SCHOOL OF CIVIL ENGINEERING

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FHWA/IN/JHRP-87/1

Final Report

PILE CAPACITY PREDICTIONS USING STATIC AND DYNAMIC LOAD TESTING

Ahmad Amr Darrag

PURDUE UNIVERSITY

JOINT HIGHWAY RESEARCH PROJECT

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by

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Joint Highway Research Project

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in cooperation with the

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and the

U.S. Department of Transportation
Federal Highway Administration

The opinion, findings and conclusions expressed in this publication are those of the author and not necessarily those of the Federal Highway Administration.

Purdue University
West Lafayette, Indiana
February 3, 1987

FINAL REPORT

PILE CAPACITY PREDICTIONS USING
STATIC AND DYNAMIC LOAD TESTING

To: H. L. Michael, Director
Joint Highway Research Project

From: C. W. Lovell, Research Engineer
Joint Highway Research Project

February 3, 1987

Project: C-36-36P

File: 6-14-16

Attached is a Final Report on the study, "Computational Package for Predicting Pile Stress and Capacity". This report is written by Ahmad Amr Darrag of our staff, who worked under my supervision.

The report is a comprehensive synthesis of pile analysis and design technique in several parts: (1) static pile load tests; (2) dynamic measurements made during pile driving; and (3) residual stresses induced in the pile and adjacent soil by pile driving. These chapters have already had one review by the IDOH and the division office of the FHWA.

As a result of this study Mr. Darrag has made definite recommendations to the IDOH with respect to: (1) how pile load tests should be run and interpreted; (2) why the IDOH should begin making and using dynamic measurements; and (3) how residual stresses can be estimated and used in pile foundation analysis and design.

Although the report is lengthy (± 300 pages), it is recommended that it be printed in its entirety.

The report is submitted for review, comment, and acceptance in fulfillment of the referenced study.

Respectfully submitted,

CW Lovell

C. W. Lovell
Research Engineer

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16. Abstract

Recommendations regarding the most reliable and economical load test methods needed by highway agencies for the design and control of bridge piles are introduced. The report contains an intensive review of the state-of-the-art of the loading equipment and instrumentations for the measurement of load and deformation.

A review of the state-of-the-art of dynamic measurements and their potential uses is included. Recommendations are made concerning the necessary equipment and the associated costs.

The phenomenon of residual stresses due to pile driving is discussed in detail. A simple approach was developed for the prediction of such stresses to make better predictions of pile capacity and to facilitate better interpretation of load results.

A computer program, named PPILENF, was developed for the purpose of predicting additional pile loads due to negative skin friction. A complete user's manual is given, including several illustrative examples.

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Pile, static load tests, quick testing methods, dynamic measurements, residual stresses, negative skin friction	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161

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LIST OF SYMBOLS AND NOMENCLATURE

A	= cross-sectional area of the pile
A_c	= area of concrete in a combined pile cross section
A_s	= area of steel in a combined pile cross section
$A(t)$	= pile acceleration as a function of time
a_r	= residual stress percent
b_c, b_r, b_m	= factors describing shape of residual load distribution
D	= pile embedment length
E	= elastic modulus of the pile
E_c, E'_c	= elastic moduli of concrete and steel, respectively
E_s	= elastic modulus of soil
$F(t)$	= force at the pile top as a function of time
F_c, F'_c	= interaction correction factors due to the effect of the reaction system
j_c	= damping constant
k_s	= coefficient of lateral earth pressure for piles
k'_τ	= unloading stiffness in friction
L	= pile length
l_2, l_3	= lengths for rods (2), (3) of the tell-tales, respectively
M	= mass of the piles
m	= skin friction percent

m_m = measured values of m

N_q = bearing capacity factor

N_{side} = average SPT blow count within the shaft length considered

P = load at the pile head

P_2 = load reaching the pile at the level of toe of rod (2) of the tell-tale system

P_3 = load reaching the pile at the level of its toe

p = pile perimeter

Q_p,Q_s = tip and shaft pile loads, respectively

Q_{PR} = residual tip load

Q_{RMAX} = maximum residual shaft load

q_{res} = residual point stress

R(t) = pile total capacity

$R_d(t)$ = pile dynamic capacity as a function of time

$R_s(t)$ = pile static capacity as a function of time

s_m = measured settlement

s'_m = measured settlement relative to reaction piles

t = time

t_x = time after which a "blip" is observed for a pile with a crack

v(t) = pile velocity as a function of time

Z_{cr} = depth at which Q_{RMAX} occurs

Z_m = depth of point of inflection on residual load distribution

α_N = factor giving the reduction in the number of blows due to the existence of residual stresses

α_S = factor giving the increase in pile stresses due to the existence of residual stresses

β_m, β'_m = factors that introduce the effect of skin friction percent on the residual stress percent

Δ = distance from the pile top at which a crack exists

Δ_L = elastic shortening over length (L)

Δ_2, Δ_3 = elastic shortening of the pile between the pile head and rod (2), and the pile head and rod (3), respectively, for the tell-tale system

Highlight Summary

Static pile load testing is the most reliable technique for pile capacity estimation and for quality control. The main disadvantage of these tests is the cost involved. Recommendations regarding the most reliable and economical load test methods are introduced in this report. It is recommended that the IDOH use these techniques routinely in the jobs involving pile foundations. The state-of-the-art for loading systems and instrumentation for deformation measurements is described as well.

It has been proven that the performance of the pile and driving system during driving cannot be monitored successfully without the use of dynamic measurements. These measurements, together with the wave equation analysis, can be used for many purposes such as pile capacity prediction, observation of hammer performance, observation of pile performance and integrity, checking the efficiency of the driving elements, quality control and other things. One of the purposes of this report is to familiarize the IDOH with the subject of dynamic measurements, illustrate their uses, and review some of the work that has been done using them. It is recommended that the IDOH acquire the necessary equipment and prepare the trained personnel required for their operation to utilize the dynamic measurements routinely in pile foundation jobs.

Residual stresses accumulating during pile driving have a very important effect on the pile capacity prediction and the interpretation of static load test results. The report examines this phenomenon in detail, including all the factors involved and the methods that have been suggested thus far for the residual stresses prediction. A new method has been developed for the prediction of such stresses. This method, in addition to being relatively simple, proved to give predictions that compare well with actual measurements.

In many situations, negative skin friction has a very important effect on the design of the pile foundation. A computer program PPILENF has been developed in order to predict additional pile loads due to negative skin friction. It is recommended to use this program for the problems involving negative skin friction to avoid serious prediction errors for long-term problems.

CHAPTER 1

INTRODUCTION

The analytical determination of pile capacity was discussed in the interim report by Tejidor (1984). It is essential to complete the subject of pile capacity prediction by discussing pile load tests. These tests provide the best evidence of pile capacity and serve as a reference of accuracy for other load prediction techniques.

The judicious use of load tests can lead to substantial economic benefits. The costs of performing pile load tests are relatively small compared to the substantial savings that may be achieved by their use. However, many organizations remain reluctant to use them because of cost and time delay during construction. Hence, it is important to improve the test procedures so as to save money and time.

One of the main objectives of this report is to form a specification for the best method available for performing load tests and to recommend its use by the IDOH. In order for the report to be most valuable, the state-of-the-art of pile load testing is described in some detail. Emphasis is placed especially on axial compression load tests, since they are the most common type for pile testing. Other types of load tests such as uplift, lateral and torsional tests are described as well. Loading systems and instrumentation for deformation measurements are described and discussed for each type of test. The major methods available for performing axial compression load tests are described in detail as well as the interpretations of their results (e.g., ultimate loads, allowable working loads, settlement behavior, ..., etc.).

The quick load tests methods have proven to correlate well with the classical longer-term testing procedures. Since the short-term tests are more economical, their use can be economically justified on a greater variety of projects. As these are used and interpreted, it is possible to gain greater confidence in static and other load prediction methods.

In spite of the recent developments that resulted in more accurate methods of predicting pile capacities; improved methods of construction control; and the use of highly specialized methods and equipment for driving, some uncertainties still exist. Thus far, static pile load tests have proven to be the best way to obtain relatively accurate information regarding the static capacity of the pile. On the other hand, the performance of the pile and driving system during driving cannot be monitored successfully without the use of dynamic measurements. These measurements have been used by many investigators and organizations to examine the pile driving procedure. Such investigations have shown that the dynamic measurements, together with the wave equation analysis, can be used for many purposes such as pile capacity prediction, observation of hammer performance, observation of pile performance and integrity, checking the efficiency of the driving elements, quality control, and other things. By using such measurements, the number of necessary static load tests can be reduced and consequently much money can be saved. One of the purposes of this report is to introduce the subject of dynamic measurements, illustrate their uses, and review some of the work that has been done using them. This is intended to familiarize the IDOH with the subject and to introduce it as a valuable and necessary link in the chain of the design and construction of pile foundations.

An important factor that has a considerable effect on the pile capacity prediction and the interpretation of load tests is the existence of residual stresses in the pile due to driving. These stresses have a very important effect on the distribution of loads along the pile shaft and beneath the tip. The report describes an explanation for such a phenomenon, discusses the factors affecting it using some parametric studies, and reviews the methods that have been suggested thus far for the residual stresses prediction. Based on these studies a new procedure is developed for such prediction. This procedure is introduced by means of easy-to-use charts and equations, with the help of some illustrative examples. Predictions made by this technique are compared with actual measurements and good agreements are proven.

The subject of negative skin friction was discussed in detail by Tejidor (1984). Further discussion is given in this report. A computer program PPILENF for the prediction of additional pile loads due to negative skin friction was developed at Purdue. This program is introduced in this report, together with a user manual, input forms and some illustrative examples. It is recommended to use this program for the problems involving negative skin friction to avoid serious prediction errors and long-term problems.

CHAPTER 2

PILE LOAD TESTS

2.1 Introduction

2.1.1 Purpose of Pile Load Tests:

Pile load tests are expensive and can be quite time consuming. In many cases, prior experience combined with adequate sub-soil data and sound judgment, can preclude the need for pile testing, especially if the pile design load is relatively low.

On the other hand, for large projects or for high capacity piles, load testing may be necessary. Such tests can result in substantial savings in foundation costs, which can more than offset the investment in the test program.

Fuller and Hoy (1970) divided the purposes for which pile load tests are made into two main categories. These are either: (1) to prove the adequacy of the pile-soil system for the proposed pile design load, or (2) to develop criteria to be used for the design and installation of the pile foundation.

The first category, i.e., routine pile load testing, is often the decision of the foundation engineer, but may be required by the general specification or building code for a certain type of construction. The data obtained from this category should be sufficient to convince the building authorities that the pile is adequate to support the design load (Lambe and Whitman, 1979). Poulos and Davis (1980) cited four reasons for carrying out routine load tests:

1. To serve as a proof test to ensure that failure does not occur before a selected proof load is reached, this proof load being the minimum required factor of safety times the working load.

2. To determine the ultimate bearing capacity as a check on the value calculated from dynamic or static approaches, or to obtain backfigured soil data that will enable other piles to be designed.

3. To determine the load settlement behavior of a pile, especially in the region of the anticipated working load. These data can be used to predict group settlements and settlements of other groups.

4. To indicate the structural soundness of the pile.

According to Fuller and Hoy (1970), the decision to perform the second category of tests, i.e. an advanced test program to develop design criteria, is usually made jointly by the owner and the foundation engineer. This decision is based on the scope of the project and the complexities of the foundation conditions.

The prime objective of a test program is to produce data to determine the most economical and suitable pile foundation, including the pile types to be used, the most efficient or highest working load for each type of pile, the required length for each type of pile, and the installation methods necessary to achieve the desired results.

2.1.2 Planning the Test Program:

Planning for pile testing is necessary, especially if the test program is conducted to develop design criteria. The first step is a detailed review of the subsoil data, in conjunction with the design requirements of the proposed structure. This review leads to the following decisions:

1. Final test data to be developed.
2. Type or types of testing to be performed.
3. Extent of the testing that will be required.
4. Special testing procedures necessary to achieve the desired results.
5. Selection of test locations.
6. Effects of soil conditions on test results and the need for any additional subsoil data.

7. Selection of the different types of piles to be tested.

8. Determination of approximate pile lengths.

9. Outline of possible installation methods to be used.

10. Preparation of the technical specifications.

The overall plan should be flexible enough to permit modifications that may be necessary as driving and testing data are produced.

Fuller and Hoy (1970) state that the following points should be covered by the technical specifications for the pile test program:

1. Prequalification of pile contractors, in the cases when the contract for the pile test program is to be awarded on the basis of competitive bidding.

2. Types of piles to be tested and maximum lengths to be furnished.

3. Size and capacity of basic pile driving equipment.

4. Driving criteria and special installation methods that may be required.

5. Types of tests and maximum testing capacity to be furnished.

6. Required testing equipment and instrumentation including calibration.

7. Testing procedures to be followed.

8. Data to be recorded and reported.

9. Payment method and schedule of bid items.

2.1.3 Types of Pile Load Testing:

Pile load tests may be static, dynamic, vibratory or explosive. However, most of these tests are of the static type. The most common type of static load testing is the compression load test, in which a direct axial load is applied to a single pile. Static load testing can also involve uplift or axial tension tests, and lateral tests applied either horizontally or perpendicular to the pile axis (in the case of batter piles). Any of these tests can be applied to pile groups consisting of vertical piles, batter piles or a combination of both.

Fuller and Hoy (1970) state that neither dynamic nor explosive testing is too reliable, and these methods are infrequently used. Vibratory testing is used only when it simulates the structure loading conditions.

2.1.4 Application of Results:

The results of the pile testing must be applicable to other piles in the site. Fuller and Hoy (1970) stated that the following conditions should exist in order for the results to be applicable for all piles of the project:

1. The other piles are of the same type, material and size as the test piles.

2. Subsoil conditions are comparable to those at the test pile locations.

3. Installation methods and equipment used are the same as or comparable to those used for the test piles.

4. Piles are driven to the same penetration or resistance or both as the test piles, to compensate for variations in the vertical position and density of the bearing strata.

The application of the results of the advance test program to the foundation design and specification can often produce substantial savings in information costs. Although, as a practical measure, the test results would lead to the selection of a single design load, the requirements for various types of piles as to size, length, shape, weight per foot, installation methods and driving requirements could vary over a rather wide range. These differences should be reflected in the specifications and, in turn, will be reflected in the alternative costs to produce the most economical foundation for the conditions involved.

Because of the time effects and the group action, the results of a static load test are not always easy to interpret.

It should be noted that the observed settlements made at the top of the pile may not necessarily indicate downward movement of the pile into the ground. Where high load tests are performed, the possibility of local failure of the pile above the ground surface, or crushing of the ground under the test plate, should be recognized as possible factors contributing to observed settlements.

Poulos and Davis (1980) emphasized that in many cases, the results of a test on a single pile cannot be directly extrapolated to predict the behavior of pile groups or other piles. Chellis (1961) stated that a pile load test can determine only the ultimate bearing capacity, and not the settlement characteristics of the pile group. He emphasized that the settlement computations are a separate matter, and the subject of soil mechanics calculations. In addition, it is impossible to evaluate tests unless adequate boring records present a complete picture of the underground at or close to the test pile. Chellis (1961) further pointed out that the volume of soil influenced by a single pile is much less than that of a large group, so the influence of deep-seated compressible layers may not be apparent in a pile load test, although such layers may critically affect the behavior of a group. Poulos and Davis (1980) then concluded that the pile load tests should be accompanied by detailed site investigations to define accurately the entire soil profile.

2.2 Axial Compression Load Tests

2.2.1 Loading and Instrumentation:

Most of the existing methods to perform the axial compression load tests use the same type of loading arrangements and pile preparation. A square cap is cast onto the head of a concrete pile with its underside clear of the ground surface. Steel piles are trimmed square to their axis and a steel plate is welded to the head, stiffened as necessary by gussets.

2.2.1.1. Loading Systems:

Whitaker (1976) summarized the most common methods used for providing the load (downward force) on the pile to be tested as follows:

1. A platform is constructed on the head of the pile on which a mass of heavy material (a kentledge) is placed. This is illustrated in Fig. (1). The load must be placed with care to obtain an axial thrust. Safety supports in the form of wedges or packings a little distance below the platform are needed on which the platform can rest to prevent the load toppling if the platform comes out of level as the pile settles.

   Whitaker (1976) indicated that this method is cumbersome and is used only in exceptional cases.

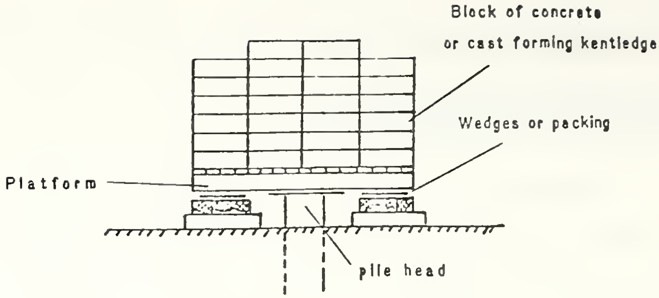


Fig. 1 Arrangement for Carrying Kentledge Directly on the Pile Head. (from Whitaker, 1976)

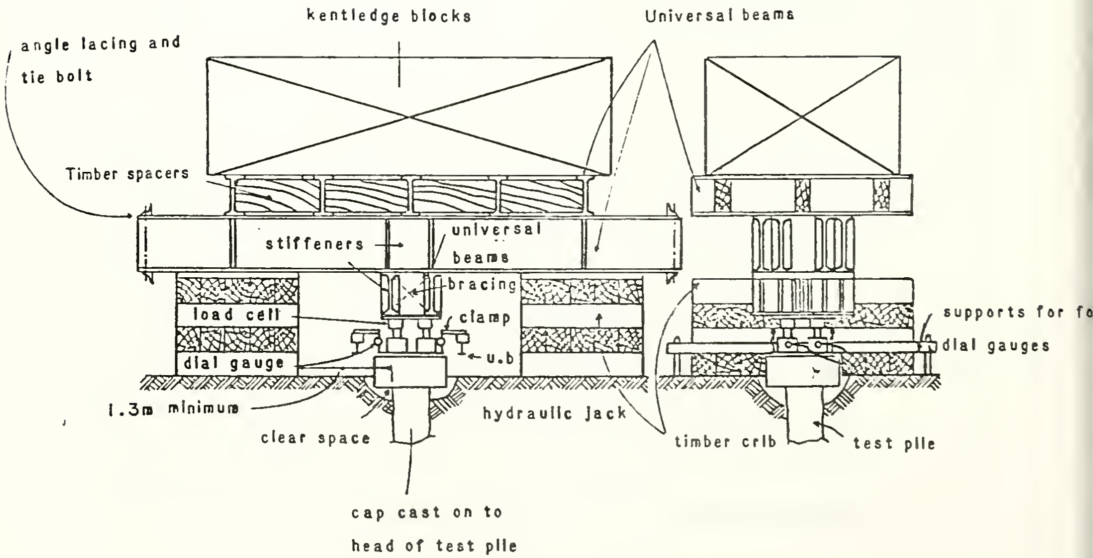


Fig. 2 Testing Rig for Compressive Test on Pile Using Kentledge for Reaction (From Tomlinson, 1977)

2. A bridge, carried on temporary supports, is constructed over the test pile and loaded with a kentledge. The ram of a hydraulic jack, placed on the pile head, bears on a crosshead beneath the bridge beams, so that a total reaction equal to the weight of the bridge and its load may be obtained. This is shown in Fig. (2). Whitaker (1976) recommended that the supports be more than 1.25 m (4 ft) away from the test pile, to minimize the effect of the supports on pile settlement.

3. Anchor piles capable of withstanding an upward force are installed on each side of the test pile, with a beam tied down to the heads of the anchor piles and spanning the test pile. In testing piles installed for the actual structure (rather than special test piles), it is often convenient to test an interior member of a group in this manner (Poulos and Davis, 1980). A hydraulic jack on the head of the test pile produces a reaction against the underside of the beam (Fig. (3)). This method is called the "boot strap" method. Whitaker (1976) recommends that any anchor pile should be at least three test pile diameters distant from the test pile, center to center, and in no case less than 1.50 m (5.0 ft). For piles with enlarged bases, the spacing should be the greater of

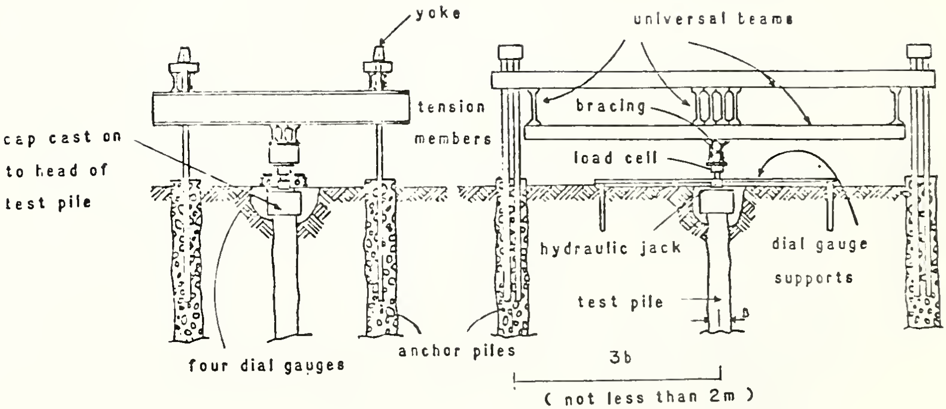


Fig. 3 Testing Rig for Compressive Test on Pile Using Tension Piles for Reaction (From Tomlinson, 1977)

twice the base diameter or four times the shaft diameter of the test pile. Poulos and Davis (1980) stated that even with these spacings, considerable interaction between the anchor piles and the test pile may occur. This will produce an inaccurate indication of the settlement of the pile (the measured value would be less than the correct value).

4. Ground anchors that usually transfer the reaction to stiffer strata below the level of the pile tip (Fig. (4)). Because the upper portion of an anchor cable does not usually transfer load to the soil, ground anchors can be placed closer to the test pile than can reaction piles.

In method (1), the applied load is determined by weighing the platform and the material placed on top of it. In the other three methods, Whitaker (1976) recommends the use of a load measuring device (e.g. a load cell or pressure capsule). If this is not possible, the load on the jack ram may be calculated from the hydraulic pressure of the fluid in the jack. However, if this method is used, the jack with its pressure gage should be calibrated in a testing machine under a full cycle of loading. Also, two pressure gages with different ranges may be required to obtain accurate measurements. The pressure gages and the jacks should be periodically calibrated and certified accurate to

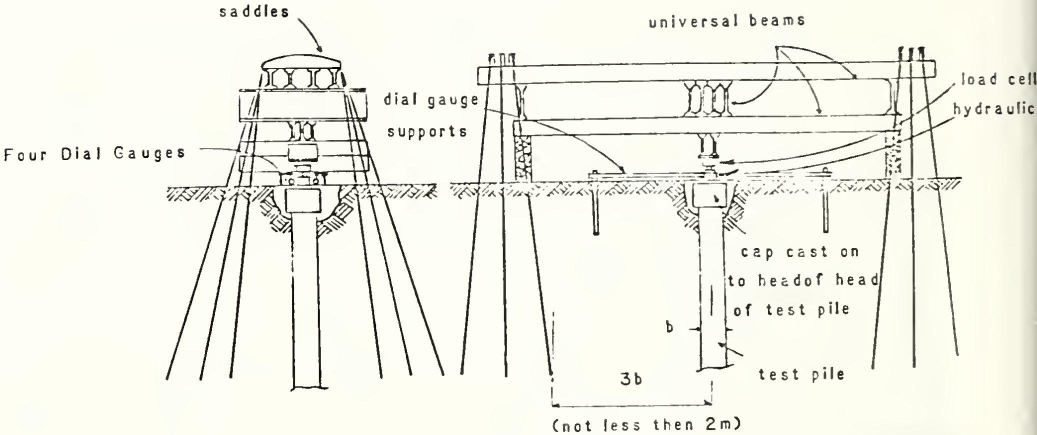


Fig. 4 Testing Rig for Compressive Test on Pile Using Cable Anchors for Reaction (From Tomlinson, 1977)

within five percent. Friction caused by corrosion and wear of the jack ram, and aging of the sealing ring can cause large errors. Friction can be reduced to an acceptably low level only by maintaining the jack in good condition. It is also necessary to avoid eccentricity of loading, or tilting of the surface against which the ram bear. Hence, care must be taken to make certain that the loading surfaces of the reaction beam and pile or shaft are parallel and that the piston is perpendicular to both. The possibility of having eccentric loads on the piston can be minimized by using steel plates or spherical leveling blocks between the piston and reaction beam, and leveling plates on top of the pile or shaft (Butler and Hoy, 1977). When testing drilled shafts or large diameter piles, it is generally necessary to use more than one hydraulic jack. When using more than one jack, each should have the same rated capacity and be from the same manufacturer. All jacks used should be connected to a common manifold and pressure gage with pressure supplied by one hydraulic pump. A hand operated pump may be used. However, an air operated pump significantly increases the efficiency of the operation. Corrosion is prevented and ram friction considerably reduced by chromium plating and grinding the cylindrical surface of the ram.

Fuller and Hoy (1970) stated that the use of hydraulic jacks has several advantages. For example, it is the only practical way to apply load-unload-reload cycles, and hydraulic jacks are more suitable for uplift tests, lateral tests and tests on batter piles.

Regardless of the method of load application, the load should be kept constant under increasing pile deflection. For direct loading this presents no problem, but when hydraulic jacks are used, this can be accomplished by activating the jack pump with a compressed gas control system.

The combined weight of the kentledge and reaction girders, or the calculated resistance capacity of tension piles or cables must be greater than the jacking force required. In the case of kentledge loading, Tomlinson (1977) suggested that the combined weight should be about 20% greater than this force. Cable anchorages or tension piles should have an ample safety factor against uplift. The former can be tested by stressing the anchors after grouting them in. If there is any doubt about the uplift capacity of tension piles, a test should be run to check the design assumptions. Increased capacity of tension piles in clays can be obtained by under-reaming them (see Tomlinson (1977)).

Restraint by a pair of anchors from a single pile to each end of the reaction girder is not a good practice, as it can cause dangerous side sway of a deep girder. Tomlinson (1977) recommends that the piles or anchor cables should be placed in pairs at each end of the girders as shown in Figure (3), (4).

Permanent piles can be used as anchorages for load tests on working piles, but it is unwise to use end bearing piles for this purpose, since as the skin friction is low, the pile may be lifted off its seating. When using tension piles, special threaded anchor bars extending above the pile head should be cast into the piles for attachment to the reaction girders. It is not advisable to weld such bars to the projecting reinforcing bars because of the difficulty in forming welds sufficient to resist the high tensile forces involved.

Tomlinson (1977) recommends that raking piles be tested by a reaction from kentledge or tension piles, since the horizontal component of the jacking force cannot be satisfactorily restrained by a jacking system. He stated that cable anchors inclined in the same direction as the raking piles can be used, but it is preferable to determine the ultimate or allowable loads on raking piles by installing special vertical piles for loading tests.

Tomlinson (1977) recommends that the hydraulic jack have a nominal capacity which exceeds by 20% or more the maximum test load to be applied to the pile. This is necessary in order to avoid heavy manual pumping effort when nearing maximum load, and to minimize the risks of any leakage of oil through the seals. For high capacity piling tests, much heavy manual effort is saved by providing a mechanical pumping unit. The ram of the jack should have a long travel where piles are being loaded near to the failure condition. This avoids the necessity of releasing oil pressure and repacking with steel plates above the ram as the pile is pushed into the ground.

Where piles are installed through fill or soft sensitive clay, these layers give positive skin resistance to the test pile, whereas they may affect the permanent piles by negative skin friction. It may therefore be desirable to sleeve the pile through these layers. This can be done by using a double sleeve arrangement as shown in Fig. (5). Alternatively, Tomlinson (1977) suggested that the outer casing can be withdrawn, after filling the anchor space between it and the steel tube encasing the test pile with a bentonite slurry.

Some suggested arrangements for applying test loads to pile groups are illustrated in Fig. (6), (7) and (8).

2.2.1.2 Measurement of Settlement of Pile Head:

Usually, the basic information obtained from the pile load test is the settlement of the pile head under the test load. This can be measured optically by means of a surveyor's level reading onto graduated scales fixed to the pile in four positions (Tomlinson, 1977). If the scales are calibrated in 0.5 mm (0.02 in.) intervals, the movement of the pile can be measured to an accuracy of ± 1 mm (0.04 in.) which is sufficient in most cases. Fuller and Hoy (1970) stated that it is quite useful to use measurements with the level and rod (or scale) as a secondary or backup system to check other measuring systems.

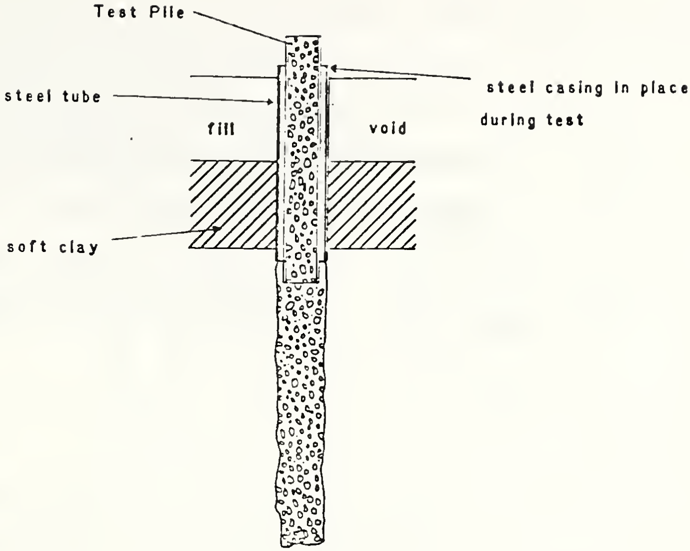


Fig. 5 Method of Sleeving Test Pile to Eliminate Skin Friction Through Fill (from Tomlinson , 1977)

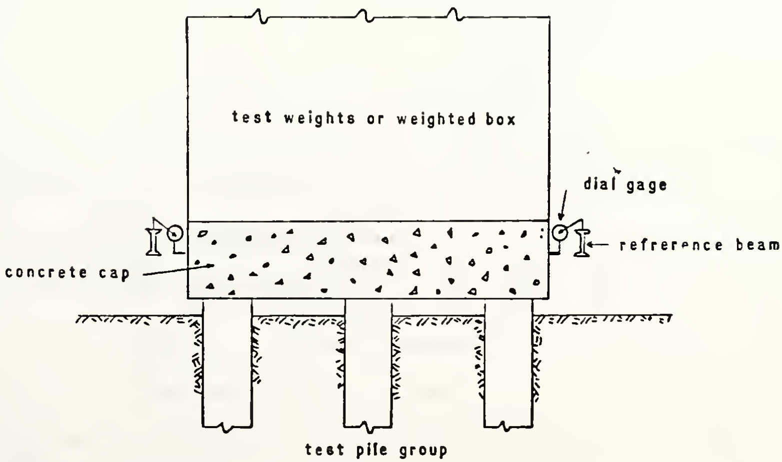


Fig. 6 Arrangement for Applying Tests Loads Directly on Pile Cap for Group Tests (ASTM, 1981)

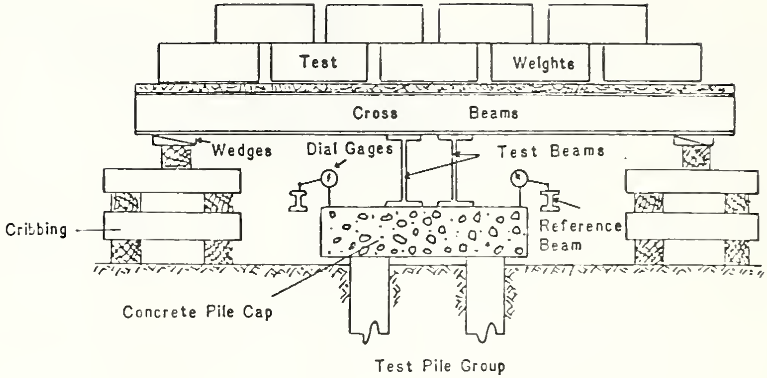


Fig. 7 Possible Arrangement For Applying Load Test To Pile Group Using Weighted Platform (ASTM , 1981)

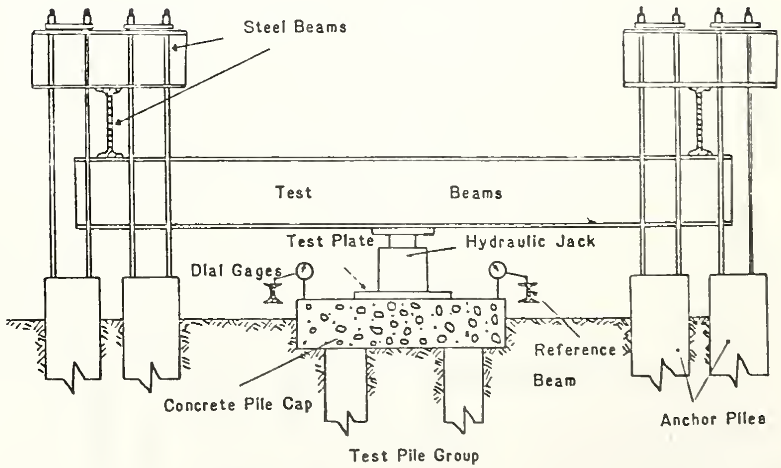


Fig. 8 Typical Arrangement For Applying Load Test To Pile Group (ASTM, 1981)

Direct readings of the pile movement can be made by using the mirror, scale and wire method. A measuring scale is fixed to a mirror which in turn is attached directly to the pile. A taut wire passing in front of the scale permits direct readings of pile movement. Consistent scale readings are obtained by aligning the wire and its image in the mirror. The wire can be kept taut by a weight and pulley system or by springs.

The most common method for measuring the pile movement is with dial gages mounted on an independent support system, and with gage stems bearing against the top of the test plate or an angle iron attached to the sides of the pile (Fig. (9)). Fuller and Hoy (1970) suggest that at least two dial gages mounted on opposite sides of the pile should be used to compensate for possible tilting or lateral movement of the pile under load. Tomlinson (1977) recommends placing a dial gage on each of four reference points on the pile head.

Sometimes a gage sensitivity of 0.1 mm (0.004 in.) is specified. Tomlinson (1977) stated that this order of accuracy is not realized in practice, since wind, temperature effects and ground vibrations can cause the datum frame to move by much more than 0.1 mm. Fuller and Hoy (1970) also stated that with ultra sensitive dial gages (0.001 in.), it is often impossible to meet some of the specification requirements, such as "...until settlement

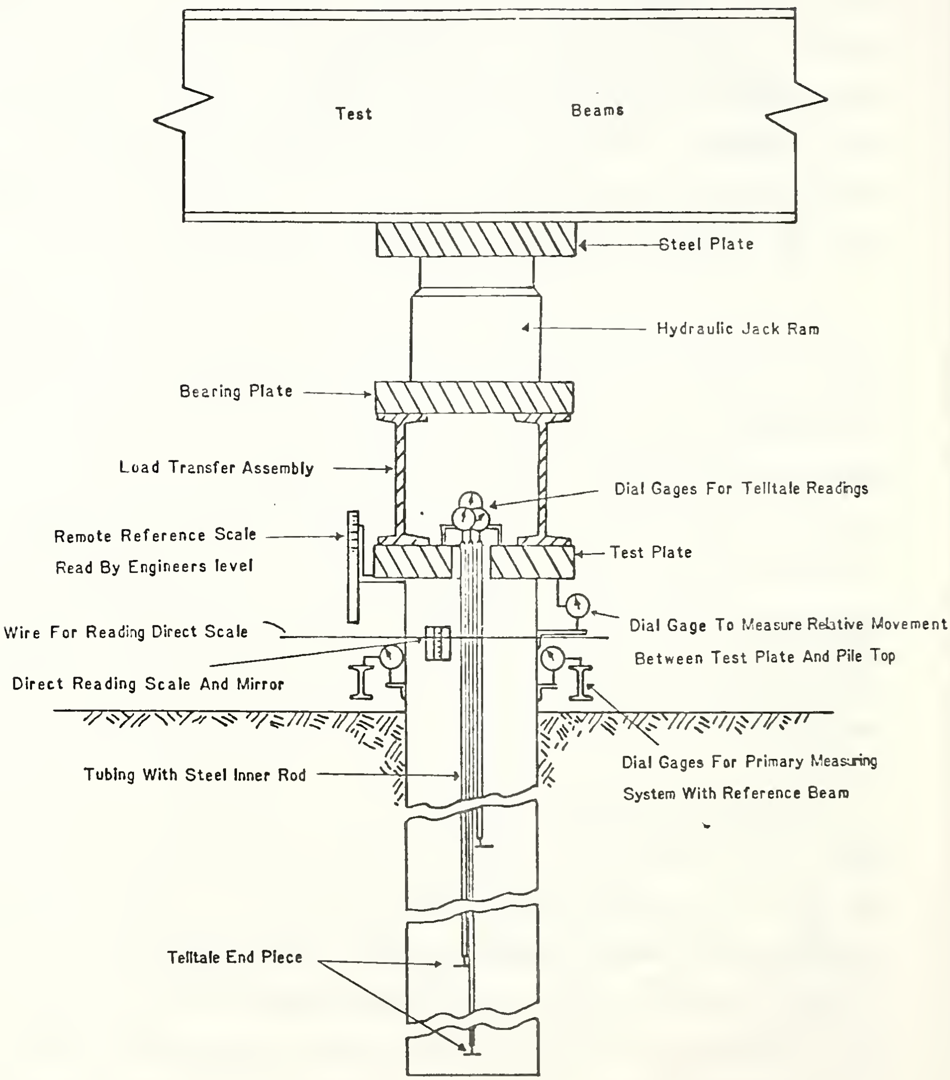


Fig. 9 Typical Setup For Measurement Of Pile Displacements. (From Vesic, 1977)

stops". They suggested that gages reading to 0.01 in. (0.25 mm) produce sufficient accuracy to meet the normal settlement criteria. However, Tomlinson (1977) recommended the use of a precision of 0.004 in. (0.1 mm) while adding each increment of jacking force, since the time settlement curve can then be plotted accurately and the rate of decrease of movement is readily obtained. Fuller and Hoy (1970) also recommended that when the instrumentation for a compression test is set up, it is often advisable to mount dial gages to measure lateral movements of the pile under test. Such movement could be due to eccentric loading, and contribute to the apparent vertical movement of the pile head.

Tomlinson (1977) suggested another way to measure settlement. A linear potentiometer can be used to obtain the pile movements, which are read on a dial or print-out mechanism at an instrument station well clear of the pile. Tomlinson (1977) stated that this method and the optical method avoid the need for the technicians to crouch under a kentledge stack to read dial gages. With a well designed kentledge support system, the technicians should not be in a dangerous position under the stack, but the working space is usually very confined, causing discomfort and fatigue to the technicians controlling a long-duration test.

The settlement measurement system must be supported independently from the loading system, with supports protected from extreme temperature variations, effects of the test load, and accidental disturbance by test personnel. Fuller and Hoy (1970) recommend the use of a secondary or backup system in case of an accidental disturbance of the primary one, or the necessity to reset dial gages so that the continuity of data is maintained.

Poulos and Davis (1980) suggested an alternative means of settlement measurement in the case where anchor piles are used as supports for the reaction system. This is to measure the settlement of the test pile with reference to the reaction piles, i.e. by fixing a dial gage to the cross beam joining the reaction piles. This is very helpful in minimizing the error in settlement measurement resulting from the interaction between test pile,, soil and reaction piles. This error is illustrated in more detail in a subsequent section of this report.

2.2.1.3 Measurement of Pile Movements and Loads at Various Points Along the Pile:

Data on load distribution (shaft and base loads are evaluated separately) and the elastic behaviour of the pile can be obtained with strain rods (sometimes called "tell-tales") or strain gages. This type of instrumentation can be installed on almost all types of conventional piling, but more readily on cast-in-place concrete piles. Strain gages or the terminal points of strain rods can be located at various positions along the pile.

Fuller and Hoy (1970) stated that strain rods are less complicated, are less subject to malfunction, are more easily handled by field personnel, and produce direct elastic shortening data of a long gage length between the terminal point and the pile head. The proper installation of strain gages, so as to avoid malfunction and produce reliable data, is an extremely sensitive operation.

The load on the base of the pile can be measured by inserting load measuring devices in a cylindrical unit interposed between the pile base and the shaft. A typical installation consists of a ring of pillar-type load cells around the periphery of the unit, connecting to a data logger at the ground surface (Hanna, 1973).

Fig. (10) shows the position of load cells for measuring the base loads in large bored piles (Whitaker and Cooke, 1966). The difference between the load cell reading and the applied load gives the load carried by frictional resistance of the shaft.

Tomlinson (1977) described the manner of using the tell-tale system to obtain the distribution of skin friction on the shaft of long hollow section piles. This system consists of metal rods installed down the interior of the pile. The rods are terminated at various levels as shown in Fig. (11), and are free to move in guides as the pile settles under load. By means of dial gages mounted on the heads of the rods, the elastic shortening of each length of pile between the toes of the rods can be measured. Thus, the load reaching the pile shaft at the toe of each rod is given by the following expressions:

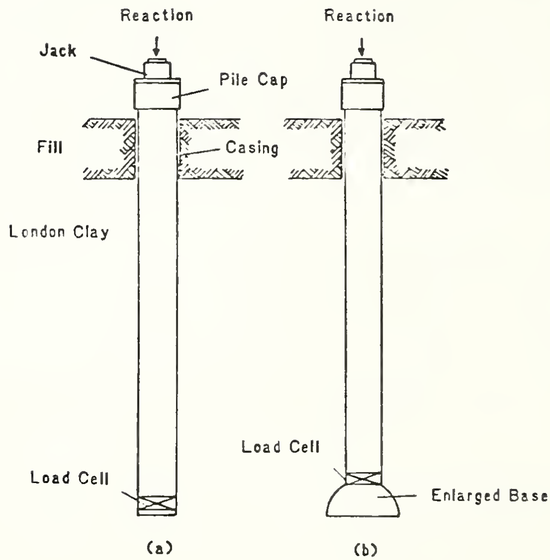


Fig 10 The Positions Of Load Cells For Measuring The Base Loads In Bored Piles, (a) Without Base Enlargement; (b) With An Enlarged Base. (From Whitaker, 1976)

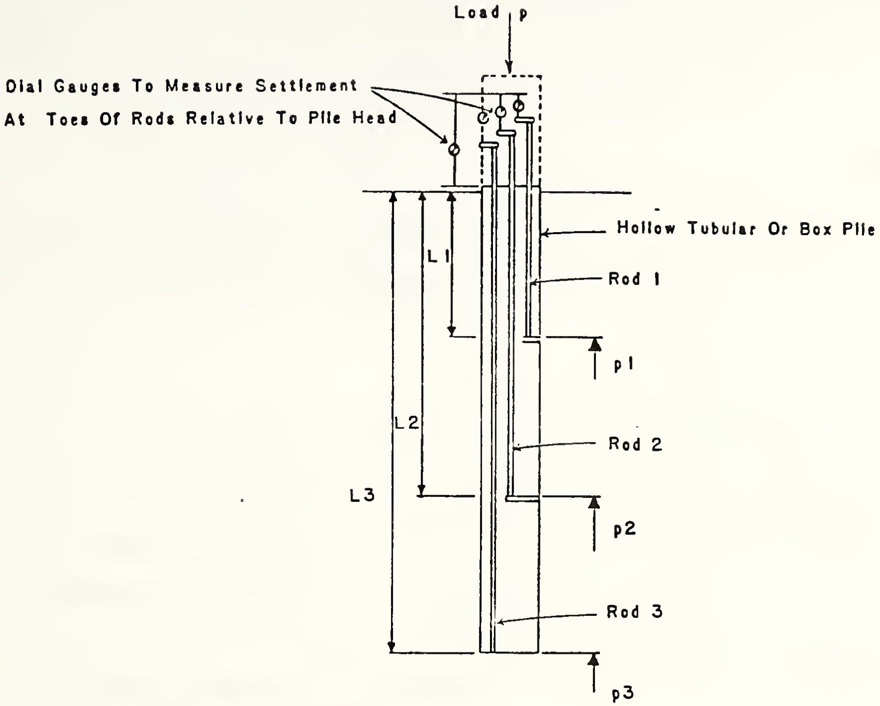


Fig 11 Use Of Rod Strain Gauges To Measure Load Transfer From Pile To Soil At Various Levels Down Pile Shaft. (From Tomlinson, 1977)

$$P_3 = \frac{2\ AE\ \Delta_3}{l_3} - P \qquad (2.1)$$

$$P_2 = \frac{2\ AE\ \Delta_2}{l_2} - P \qquad (2.2)$$

where:

P_3 = load reaching the pile at the level of the pile toe.

P_2 = load reaching the pile at the level of the toe of rod (2)

P = load on the pile head

A = cross-sectional area of the pile shaft

E = elastic modulus of the material of the pile

Δ_3, Δ_2 = the elastic shortenings of the pile between the pile head and rod (3), and the pile head and rod (2), respectively

l_3, l_2 = the lengths of rods (3), (2), respectively

The load at the toe of rod (1) is obtained in a similar manner.

Where the rods are used in the interior of a steel tubular pile filled with concrete, the elastic shortening between each length of pile is that due to the elastic modulus of the composite section. Hence:

$$\Delta_L = \frac{P}{A_s\ E_s\ (1 + \frac{A_c}{A_s} \cdot \frac{E_c}{E_s})} \tag{2.3}$$

where:

Δ_L = elastic shortening over length L

P = load on length L

A_s, A_c = area of steel and concrete in section, respectively

E_s, E_c = elastic moduli of steel and concrete, respectively

The distribution of skin friction on the pile shaft may also be measured by fixing electrical resistance strain gages onto the interior surface of a hollow steel pipe, or to a steel pipe embedded in a precast or cast-in-situ concrete pile. Gages of this type can withstand the impact of pile driving and have given satisfactory service on piles which have remained in the ground for a year or more.

Tomlinson (1977) stated that while these forms of instrumentation (either tell-tales or strain gages) are used mainly for research-type investigations, they can be adopted for the preliminary test piling stage to give useful design information at a relatively small additional cost.

Fuller and Hoy (1970) noticed that the installation of strain rods or gages results in a physical change in the cross section of the pile and thus its elastic properties. They suggested the use of a single strain rod on the pile tip, which is sufficient to provide the essential information on the elastic behavior of the pile and the basic load distribution.

The load distribution can also be approximated by driving and testing piles of different lengths. Some would be driven just short of the end bearing stratum, while others would be driven to full embedment. An uplift test might also produce approximate data on the amount of load carried by friction in a compression pile.

2.2.1.4 Residual Stresses:

The importance on pile test interpretation of residual stresses in driven piles has been emphasized by Holloway et al (1975). Compressive residual loads are likely to exist in the lower part of the pile. These stresses seem to depend on the pile-soil system only, and are independent of the impact pile driving apparatus used, (Poulos and Davis, 1980). When a residual point load remains after driving, a portion of the point bearing capacity has already been mobilized. However, if load distribution measurements are made, the gages are generally zèroed at the start of the test and the residual loads are ignored in the test interpretation. In compression load tests to failure, the measured point bearing value in such cases is only that mobilized from the start of the load test. The actual point capacity is the measured value plus the residual point load.

Conversely, in tensile load tests, the effect of residual compressive loads is to cause an apparent tensile resistance at the point. Poulos and Davis (1980) stated that while the effects of residual loads are not readily taken into account, recognition of their effects may at least resolve apparent anomalies in some load tests. The subject of residual stresses is going to be dealt with in detail in a separate chapter of the report.

2.2.1.5 Sources of Error in Settlement Measurements

Some of the loading and settlement procedures commonly used in pile load tests may lead to inaccuracies in the measurement of the settlement of a test pile. These errors arise from the effects of the supports of the loading system on the measurement settlement or the movement of the test pile. Hence a minimum distance between the test pile and supports should be specified. It is uneconomical, however, to space the supports so widely apart such that all effects are eliminated. Hence, the contribution of these effects should be calculated and allowed for in the interpretation of the test results. A theoretical examination of such inaccuracies caused by the various procedures has been made by Poulos and Mattes (1975). These errors are summarized and discussed in this section.

2.2.1.5.1. Errors Resulting from Use of Reference Beam

With this system of settlement measurement (Fig. (2)), the beam supports settle because of the loaded pile. A theoretical assessment of the resulting errors in the settlement measured may be accomplished by using the solutions for the settlement of a point on the surface of the soil caused by a loaded pile (Poulos and Davis, 1980). Following this analysis, it can be shown that:

$$S = F_c \, S_m \tag{2.4}$$

where

S = true settlement of loaded pile

S_m = measured settlement

F_c = interaction correction factor due to the effect of the beam support settlement

The correction factor F_c evaluated for a friction pile is plotted in Fig. (12)a, b for the cases of a deep layer and a pile in a finite layer (from Poulos and Davis, 1980). Fig. (12) indicates that serious errors can arise in settlement measurements on a test pile in a deep soil layer, unless each support of the reference beam is placed about 0.5 to 1.0 pile length away from the pile. In terms of the dimensionless distance r/L, the effect is more severe for shorter piles. Fig. (13) shows how the effect of the support beam movement diminishes as the soil layer thickness decreases. This error is not significant for an end bearing pile (H/L = 1). Poulos and Davis (1980) recommend that the supports be 0.3 to 0.5 pile lengths away from the test pile. Tomlinson (1977) recommends a minimum distance of 1.3 m, due to economic considerations, and in this case the correction factor

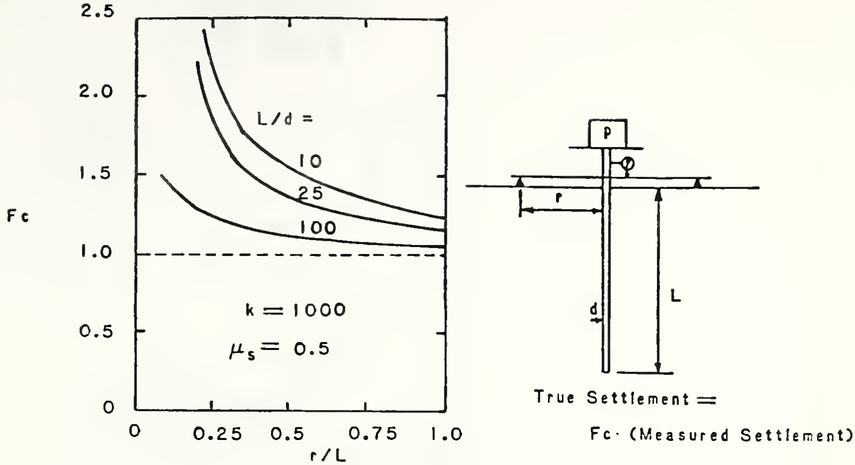


Fig. 12.a Correction Factor Fc For Friction Pile In Deep Layer Of Soil. (From Poulos and Davis, 1980)

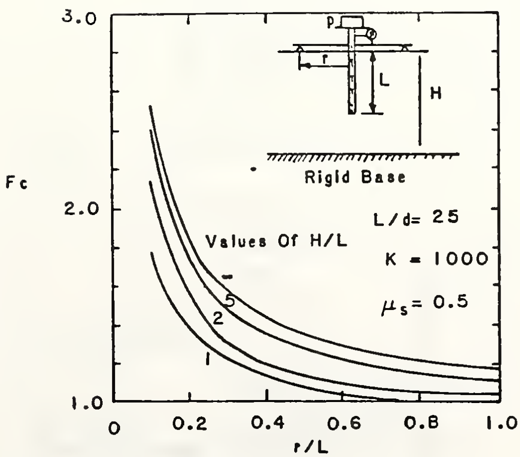


Fig. 12.b Effect Of Layer Depth On Settlement Correction Factor. (From Poulos And Davis. 1980)

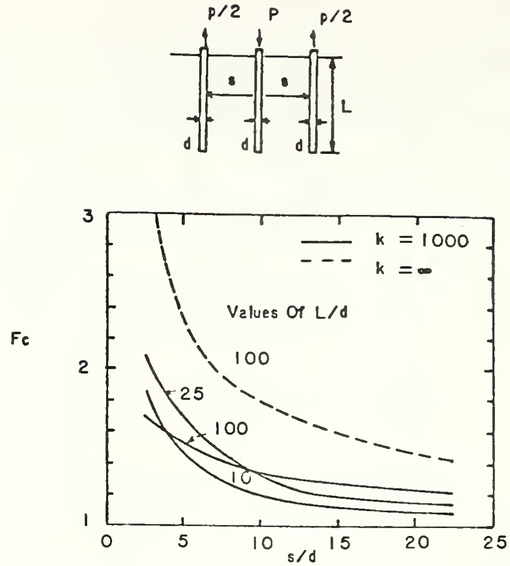


Fig. 13.a Correction Factor Fc For Floating Pile In A Deep Layer Jacked Against Two Reaction Piles. (From Poulos and Davis, 1980)

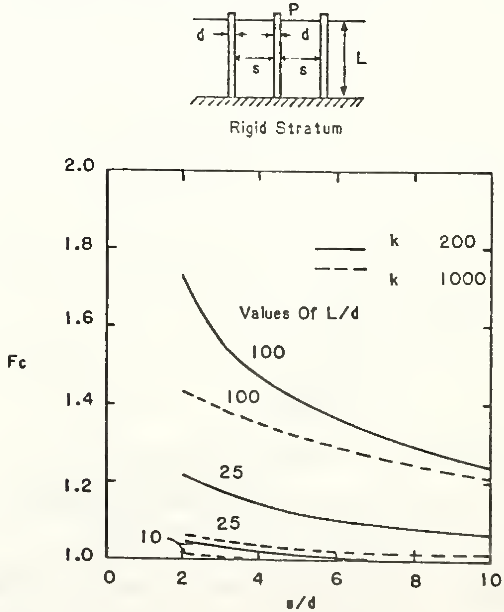


Fig. 13.b Correction Factor Fc For End Bearing Pile On Rigid Stratum Jacked Against Two Reaction Piles (From Poulos And Davis, 1980)

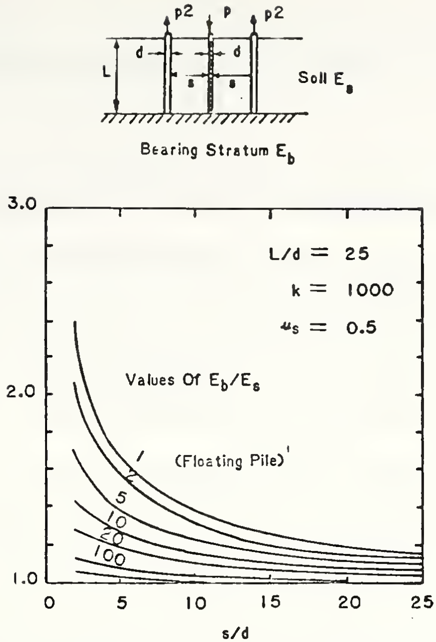


Fig. 13.c Correction Factor F_c. Effect Of Bearing Stratum For End-Bearing Pile Jacked Aggainst Two Reaction Piles.
(From Poulos and Davis, 1980)

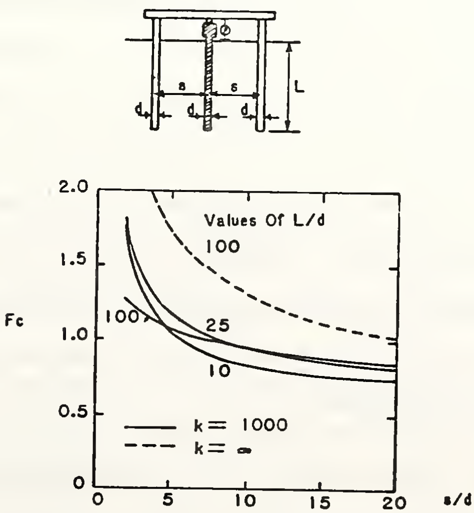


Fig. 13.d Corraction Factor F_c For Friction Pile In A Deep Layer Jacked Against Two Reaction Piles—Settlement Measured In Relation To Anchor Piles.(From Poulos And Davis, 1980)

should be applied to obtain the true settlement of the loaded pile.

2.2.1.5.2. Errors Resulting from Jacking Against Anchor Piles:

With this method of load application (Fig. (3)), the upward loads on the anchor piles cause an upward movement of the test pile because of interaction. Consequently, if the settlement of the test pile is measured with reference to a remote benchmark, the measured settlement will be less than the true settlement. A theoretical examination of this error may be made by using the pile settlement interaction solutions (Poulos and Davis, 1980). Using this analysis, a correction factor is obtained and equation (2.4) can be used. Values of F_c for various cases are plotted in Fig. (13)a, b, c. In the case of friction piles (Fig. 13.a), it can be seen that in the range of spacings commonly used (2.5 to 4 diameters), between the test and the reaction piles, the measured settlement may be one half or less of the true settlement. The error becomes larger for stiffer, more slender piles. Unfortunately, it is for such cases (long piles in very soft soils), that accurate settlement measurements may be most necessary (Roulos and Davis, 1980). However, it is not recommended that long term settlements of friction piles be obtained from load tests, since the time taken to achieve them is much longer than the desired test duration.

Fig. (13.b) shows values of (F_c) for end bearing piles resting on a rigid stratum. In this case, the interaction is generally much less for normal spacings.

The effect of relative stiffness of the bearing stratum on (F_c) is shown in Fig. (13.c). As the bearing stratum becomes stiffer, interaction decreases, and hence (F_c) decreases for a given pile spacing. However, significant errors in settlement may still occur at normal pile spacings, unless the bearing stratum has a stiffness more than about 10 times the overlying soil.

The above mentioned correction factor (F_c) is for settlement measurements with reference to a remote benchmark. To reduce this factor, an alternative means of settlement measurement is suggested by Poulos and Davis (1980). In this case, the settlement of the test pile is measured with reference to the reaction piles, i.e., by fixing a dial gage to the cross beam joining the reaction piles. The correction factor is then denoted by F_c', and can be used in the following equation:

$$S = F_c' \; S_m' \tag{2.5}$$

where:

S = true settlement of loaded pile

S_m' = measured settlement relative to reaction piles

F_c' = corresponding interaction correction factor

Values of F_c' are plotted against dimensionless spacing (s/d) in Fig. (13.d) for friction piles in a deep soil layer.

The values of F_c' are smaller than those of F_c shown in Fig. (13.a), i.e., less correction of the measured settlement is required if measurement is made with respect to the anchor piles. It must be pointed out that at large spacings, or in cases where little interaction is likely to occur between the test pile and the reaction piles, F_c' will be less than one, i.e., the measured settlement will be greater than the true settlement.

From the above discussions, it seems that measurement of the test pile settlement relative to the reaction piles have advantages over other means of settlement measurement. However, in any such pile test, measurement of the settlement by both of the alternative methods is desirable, so that a better assessment of the true settlement may be obtained (Poulos and Davis, 1980).

All the above solutions apply for a homogeneous soil stratum. The expressions in equation (2.4) and (2.5) also apply for nonhomogeneous soils, provided that appropriate values of the interaction factors are used. Poulos and Davis (1980) stated that these factors tend to be smaller for nonhomogeneous soils than for homogeneous soils. The errors involved in the test procedures will be correspondingly smaller. However, general characteristics of behaviour and variation of F_c and F_c' with spacing remain similar.

2.2.1.5.3. <u>Errors Resulting from Jacking Against Ground Anchors</u>:

The upward reaction on each ground anchor will tend to reduce the settlement of the test pile. The correction factor (F_c) used in equation (2.4) can be calculated, assuming that the effect of the ground anchor on the test pile is the same as the effect on a point located halfway along the pile. (The anchors are small in relation to the test pile.) (F_c) is plotted against dimensionless anchor spacing for various values of embedment of the anchors in Fig. (14.a). It is clear that the correction factor is much smaller in this case than in the case of jacking against anchor piles. Beyond an anchor depth of about 2L, the radial distance of the anchors from the piles has little effect on the measured settlement. In addition, the case considered in Fig. (14.a) is not likely to occur frequently in practice, since to obtain adequate load capacity, the anchors are usually secured into a stiffer layer at or below the level of the pile tip. In such cases, the upward movements caused by the anchors would be smaller, so that (F_c) will be less than indicated in Fig. (14.a).

Fig. (14.a) also generally gives an overestimate of (F_c) for an end bearing test pile bearing on a stiff layer (Poulos and Davis, 1980). Hence, for this case, (F_c) is plotted in Fig. (14.b), together with the other limiting case of a pile in a homogeneous deep layer, with anchors at the level of the pile tip (the curve for $H/L = 1.0$ in Fig. (14.a)). It can be seen that in the case where the pile bears on very stiff rock through very stiff soil ($E_b/E_s \rightarrow \infty$) ($F_c$) is extremely small, even for very closely spaced anchors.

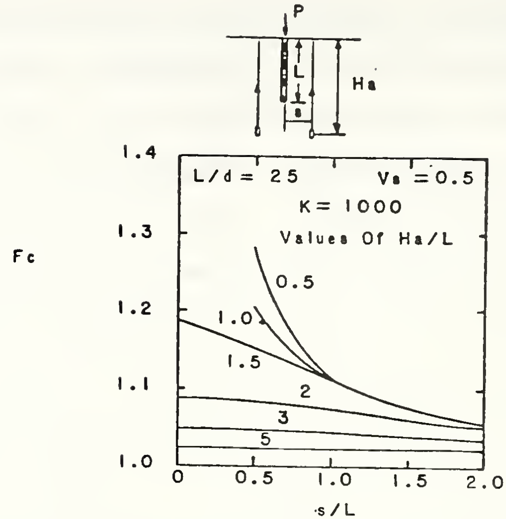


Fig. 14.a Correction Factor Fc For Friction Pile In
A Deep Layer Jacked Against Ground Anchors.
(From Poulos and Davis, 1980)

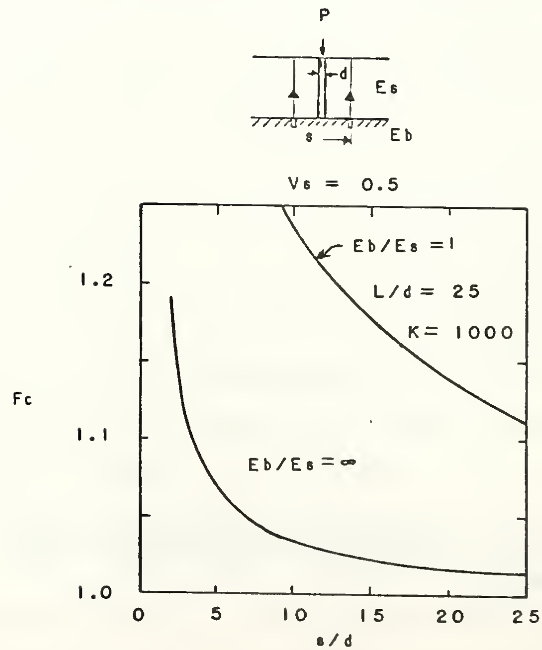


Fig. 14.b Correction Factor Fc For End-Bearing Pile Jacked
Against Ground Anchors.(From Poulos And Davis,

Fig. (14.b) indicates that in the case of friction piles in a deep homogeneous layer ($E_b/E_s = 1$), with the anchors being fixed at the level of the pile tip, the spacing between the test pile and the anchors should be as great as possible. Poulos and Davis (1980) suggested a ratio of 10 or greater. Greater spacings may be achieved by installing inclined anchors.

Comparison with the other two reaction systems shows that (F_c) for the anchor system is generally much less, i.e. less error is involved in settlement measurements when anchors are used. However, using anchors involves higher costs, since in the case of anchor piles, the neighboring piles (already in place) could be used, while special arrangements must be made to provide anchors as supports to the reaction system.

The above discussions and factors, although they may not be very accurate quantitatively, do provide an evidence of the large effect of the interaction with the reaction system on the test results. One should be very careful when interpreting the results. Many organizations prefer the kentledge method of applying the load since it does not imply any effect of interaction on the results.

2.2.2 Test Procedures

As stated earlier by Fuller and Hoy (1970), one can classify load tests, with respect to the purpose for which they are run, into two main categories. The first one is performing tests to prove the adequacy of the pile soil system for the proposed pile design load. The second one is performing tests to develop criteria to be used for the design and installation of the pile foundation. The first type of tests, i.e. most of the routine tests, are carried to one and one-half or twice the proposed design load for a single pile, or one and one-half times the design load for a pile group. Rarely can additional data be used advantageously, such as for redesign, without seriously affecting the time schedule.

On the contrary, the second type of tests, i.e. test programs that are specifically executed to produce design data, should include testing piles to failure in order to develop the most efficient design. However, Fuller and Hoy (1970) stated that this is not always essential, and definite design decisions can be reached if sufficient routine testing is done on piles of different types, sizes, shapes and lengths.

In many cases, there should be a certain time interval between pile driving and testing. This interval depends on the type of pile and subsoil conditions. For example, sufficient time should be permitted for the proper curing of cast-in-place piles before they are tested. If the pile is driven into a cohesionless sand, there may be a relaxation of the soil around the pile with a corresponding reduction in load capacity. In the case where test piles are driven into cohesive soils, it is advisable to wait several days for the soil to regain its shear strength which was reduced because of the remolding effects of pile driving. Most existing codes prescribe a minimum waiting period between driving and testing not exceeding one month. Although this requirement may be adequate for piles in relatively pervious soils, such as sands and inorganic silts, it is obviously not sufficient for piles in clay, particularly if they are of larger size. Because it may be impractical to prescribe longer waiting periods, an estimate of additional gains in bearing capacity between pile testing and application of service loads may be in order (Vesic, 1977).

A variety of test procedures have been developed for carrying out pile load tests. Among the most common procedures for compression tests are the maintained loading tests (M.L.), constant-rate-of-penetration tests (C.R.P.), method of equilibrium load tests, quick load tests, and constant settlement increment loading tests. In this section, these procedures and their interpretation are reviewed.

2.2.2.1. Maintained Loading Test:

This is the most common method of carrying out a compression load test, especially if the load settlement relationship is required.

2.2.2.1.1. Procedure:

The procedure is to apply the load in stages usually about

25% of the proposed design load, the load at each stage being maintained constant until the resulting settlement of the pile virtually ceases, before applying the next increment.

The Civil Engineering Code of Practice No. 4 (1954) proposes a rate of movement of 0.012 in./hr (0.305 mm/hr) as the limiting rate, before addition of the next increment. ASTM D1143-81 requires a rate of settlement less than 0.01 in./hr (0.25 mm/hr) or until two hours has elapsed, whichever occurs first. Poulos and Davis (1980) stated that it is doubtful whether a time interval of two hours is always adequate to ensure completion of settlement. However, taking into account that the major proportion of the settlement of a pile <u>in a load test</u> occurs as immediate settlement, they suggested that relatively short intervals between load increments should be acceptable.

ASTM D1143-81 requires that unless failure occurs first, the pile should be loaded to 200% of the anticipated pile design load for tests on individual piles, or to 150% of the group design load for tests on pile groups. It requires the load to be applied in increments of 25% of the individual pile or group design load. However, Fuller and Hoy (1970), for the sake of saving time, suggested that the increments can be larger during the early stages of the tests and, in the interest of obtaining accuracy, they should be smaller as the total load is increased.

In case the test pile or pile group has not failed, ASTM D1143-81 requires the test load to be removed any time after 12 hours if the head settlement over a one-hour period is not greater than 0.01 in. (0.25 mm); otherwise the total weight should be allowed to remain on the pile or pile group for 24 hours. After the required holding time, the test load is removed in decrements of 25% of the total test load, with one hour between decrements.

In the case of failure, jacking the pile should continue until the settlement equals 15% of the pile diameter or diagonal dimension.

The usual procedure followed in this method is to increase the load in stages until the proposed working load is reached, and then to unload and to leave the load off until the rise or rebound substantially ceases. The pile is then reloaded to the working load or the next higher stage, and the test continued to the maximum load. The unloading of the pile from the maximum load is often carried out in stages, with a pause at each stage, until rebound virtually ceases, before unloading to the next stage.

Tomlinson (1977) suggested a system of load increments for an ML test up to 1.5 times the working load. Table 1 illustrates this system.

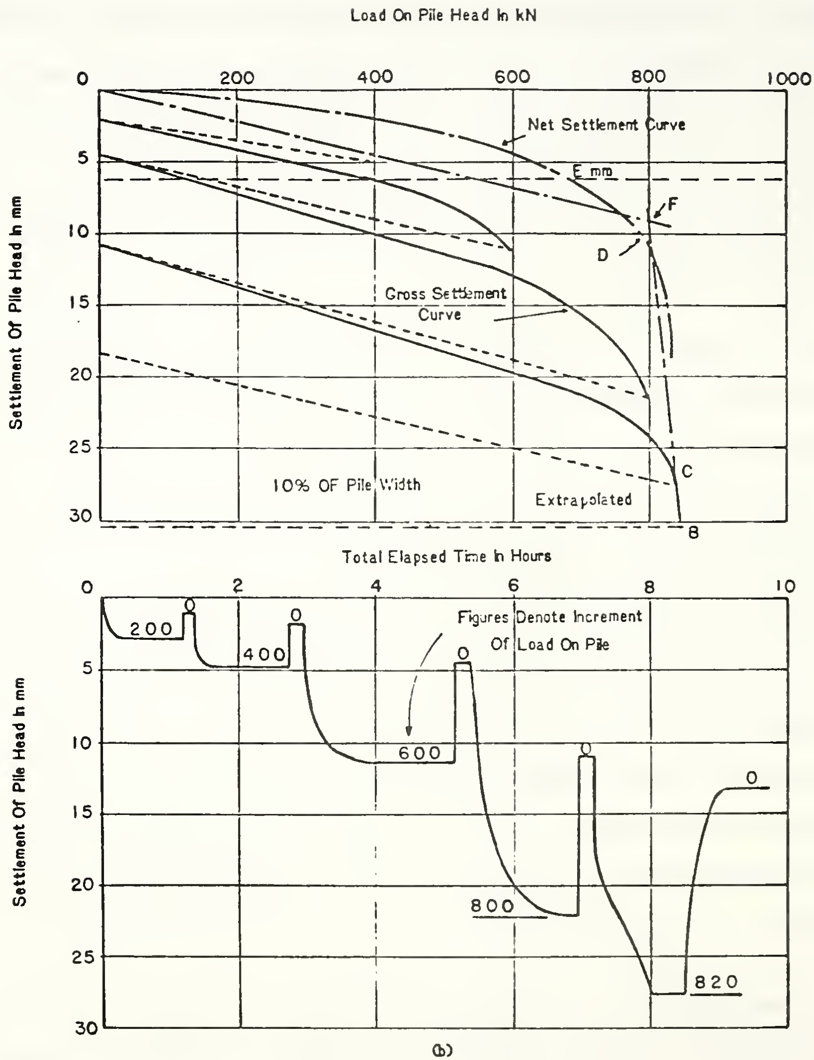


Fig. 15 Compression Load Tests On 305x305mm Pile.
Load–Settlement And Time–Settlement Curves For Pile On Stiff Clay (ML)–(From Tomlinson, 1977)

Table 1 A Suggested System of Load Increments for an ML Test (after Tomlinson, 1977)

Load as percentage of working load	Minimum time for holding load
25	1.0 hour
50	1.0 hour
75	1.0 hour
100	1.0 hour
75	10.0 minutes
50	10.0 minutes
25	10.0 minutes
0	1.0 hour
100	6.0 hours
125	1.0 hour
150	6.0 hours
125	10.0 minutes
100	10.0 minutes
75	10.0 minutes
50	10.0 minutes
25	10.0 minutes
0	1.0 hour

A similar system can be devised for a test to twice the working load. Tomlinson (1977) suggested that if it was desired to obtain the ultimate load on a preliminary test pile, it is useful to adopt the ML method for up to twice the working load, and then to continue loading to failure at a constant rate of penetration.

An alternative procedure based on constant time interval loading was suggested by ASTM-D1143-81. This procedure follows the same standards for the traditional maintained load tests, except that the load is applied in increments of 20% of the pile or group design load, with one hour between load increments. The piles are unloaded with one hour between load decrements.

A further modification of the ML test consists of returning the load to zero after each increment. This form of test is necessary if the net settlement curve is used as the basis of defining the failure load (Chellis, 1961). This procedure is discussed in detail later in this chapter.

Fuller and Hoy (1970) recommend that instrumentation readings should be taken before and after each increment of load and at sufficient intermediate intervals to define the load time deflection curves. They also recommended that when piles are not tested to failure, and after the full test load has been applied, readings are taken at least every 30 minutes.

It should be noted that if the direct loading method (using a kentledge) is used (Fig. (1), (2), (6) and (7)), the weight of the test beam(s) and the platform should be included in the first load increment. Before adding or removing load increments, the wedges should be tightened along the platform edges to stabilize the platform. Load increments should be placed or removed in a manner which avoids impact, and maintains the load balanced at all times. After each load increment has been added, the wedges should be loosened (but not removed) and kept loose to permit the full load to act on the pile as settlement occurs.

It should be noted that the load increment applied is maintained as specified before, in order to achieve a state of equilibrium between load and the corresponding settlement. Mohan et al (1967) stated that this ideal state is seldom reached. With a hydraulic jack, continuous care and occasional pumping is required to maintain the load, unless a load maintainer is available, and this is both laborious and time consuming.

2.2.2.1.2 Interpretation of Test Results:

a) Ultimate Load:

The ultimate pile capacity is defined in most geotechnical literature as the load beyond which the pile will begin to break into the ground, or expressed mathematically as $\Delta S/\Delta P \rightarrow \infty$, i.e., when the tangent to the load settlement curve becomes vertical (Chellis, 1961). However, in pile engineering practice, it is customary to select a point beyond which the rate of change increases markedly, or the settlement increment increases excessively in comparison with the load increment, to denote failure. Whitaker (1976) suggests that it is necessary to first estimate the load capacity, so that a suitable loading and reaction system may be provided, and then to define some physical event by which the ultimate bearing capacity (at failure) can be recognized when that point is reached.

Among the commonly used definitions of the ultimate load capacity, Tomlinson (1977) selected the following ones (Fig. 15):

1. The load causing a gross settlement of 10% of the least pile width (Point B), (Terzaghi, 1942).

2. The load beyond which there is an increase in gross settlement disproportionate to the increase in load (Point C). This criterion is defined more precisely by the Swedish Piling Commission as the load which gives twice the movement of the pile head obtained for 90% of the load.

3. The load beyond which there is an increase in net settlement disproportionate to the increase of load (Point D). (The net settlement is the gross measured settlement minus the elastic compression of the pile.)

4. The load that produces a plastic yielding or net settlement of 6 mm (0.25 in) (Point E).

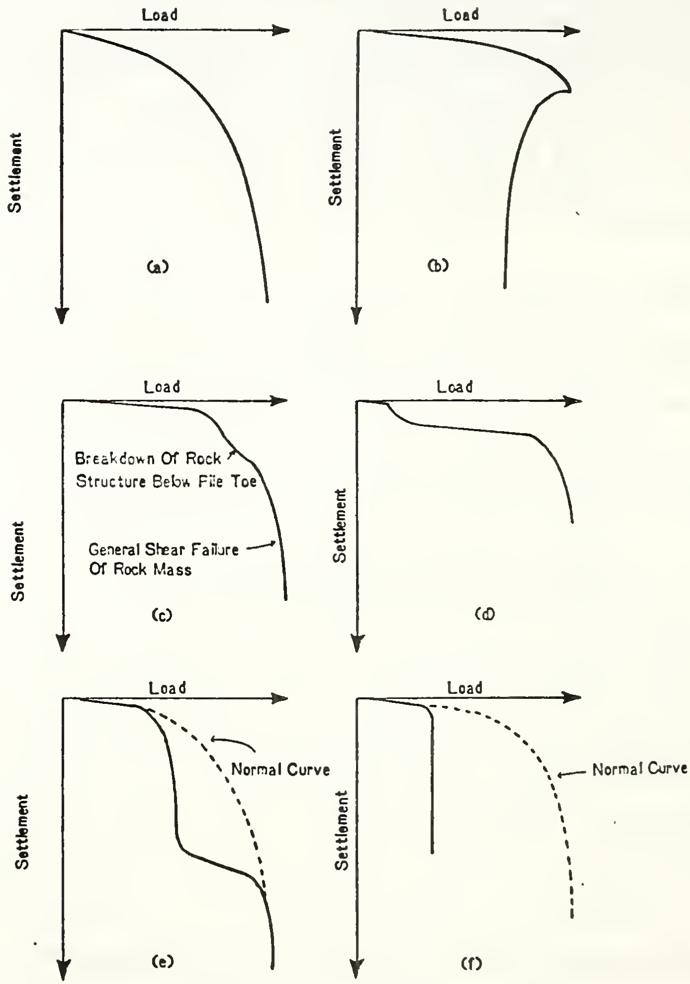
5. The load indicated by the intersection of tangent lines drawn through the initial, flatter portion of the gross settlement curve and the steeper portion of the same curve (Point F).

With experience, the load settlement curve from a compression test can be used to interpret the mode of failure of a pile. A defective pile shaft is also indicated by the shape of the curve. Some typical load-settlement curves and their interpretation are shown in Fig. (16) (after Tomlinson, 1977).

b. Empirical Methods of Working Loads:

Many arbitrary or empirical rules have been used or are contained in codes to serve as criteria for determining the allowable working load from test results. Some separate plastic and elastic deformations, others do not. (Evaluation of elastic and plastic deformation is discussed later in this chapter.) Chellis (1961) lists about seventeen rules for determining the allowable working loads. Poulos and Davis (1980) summarized a few of these as follows:

1. "The test load shall be twice the design load and shall be maintained constant for at least 24 hr., and



(a) Friction Pile In Soft–Firm Clay Or Loose Sand

(b) Friction Pile In Stiff Clay

(c) Pile End Bearing On Weak Porous Rock

(d) Pile Lifted Off Seating On Hard Rock Due To Soil Heave And Pushed Down By Test Load To New Bearing On Rock

(e) Gap In Pile Shaft Closed Up By Test Load

(f) Weak Concrete In Pile Shaft Sheared Completely Through By Test Load

Fig. 16 Typical Load–Settlement Curves For Compressive Load Tests. (From Tomlinson, 1977) until settlement or rebound does not exceed 0.22 in. in 24 hr. The design load shall not exceed one half the maximum applied load, provided that the load settlement curve shows no sign of failure, and the permanent settlement of the top of the pile, after completion of the test, does not exceed 1/2 in. (Boston Building Code).

2. Tests shall be run to 200% of the proposed load, and considered unsatisfactory if after standing 24 hr, the total settlement after rebound is more than 0.01 in. per ton of total test load (Building Laws of the City of New York).

3. Observe the point at which the gross settlement begins to exceed 0.03 in. per ton of additional load, and divide by a factor of safety of 2 for static loads, or 3 for vibratory loads (W. H. Rabe).

4. Observe the point at which the gross settlement begins to exceed 0.05 in. per ton of additional load, or at which the plastic settlement begins to exceed 0.03 in. per ton of additional load, and divide by a factor of safety of 2 for static loads, and 3 for vibratory loads (R. L. Norlund).

5. Take two thirds of the maximum test load where settlement is not excessive, and where load and settlement

> are proportional. Where the test load is carried to failure, take two thirds of the greatest load at which settlement was not excessive, and at which loads and settlement were proportionate (United States Steel Co.)."

Chellis (1961) considered rules (3) and (4) the most reasonable, although (3) may be too conservative. Poulos and Davis (1980) stated that rules such as (5) are unreliable, as various impressions of the steepness of the load settlement curves may be obtained by varying the scale of the plot. A finite limit of the change of load to change of settlement ratio is desirable.

c. Plastic and Elastic Deformations:

The total measured movement of pile head is caused by:

a. Elastic deformation of pile and soil.

b. Plastic deformation of soil.

We should be interested in the plastic deformation rather than the total one. Assuming that all the piles are of the same material, of approximately equal length, and driven into substantially similar soils, the elastic shortening will be approximately equal for all piles and thus will not contribute to differential settlements. Fig. (17) illustrates the division of total movement into elastic and plastic parts. Chellis (1961) recommended that the working load and factor of safety should be detemined using the curve of plastic deformation. To obtain this curve, cyclic loading can be used, i.e., by removing the load from the pile several times during the process of adding load increments and plotting the rebounds as shown in Fig. (17). A smooth curve could be drawn, connecting the points. By subtracting the values of this curve from those of the curve of total settlement, the elastic deformation curve may be plotted and compared with the theoretical elastic deformation line of the pile (Fig. 17). If the actual elastic curve has smaller ordinates than the theoretical at every point, this indicates that the upper undesirable strata, not selected for permanent load carrying purposes, are supporting some of the load, at least temporarily. With increasing test load, the ordinates of the actual elastic curve should eventually increase to coincide with the theoretical elastic line (Fig. 17).

The theoretical elastic deformation in the case of an end bearing pile unrestrained by friction can be computed from the formula $S = RL/AE$ and will be a straight line. For friction piles, the theoretical elastic deformation of the pile can be computed by assuming the location of the center of resistance and considering (L) as the distance down to this point (Chellis, 1961).

d. Distribution of Load to Soil:

Van Weele (1957) suggested an approximate method to get the

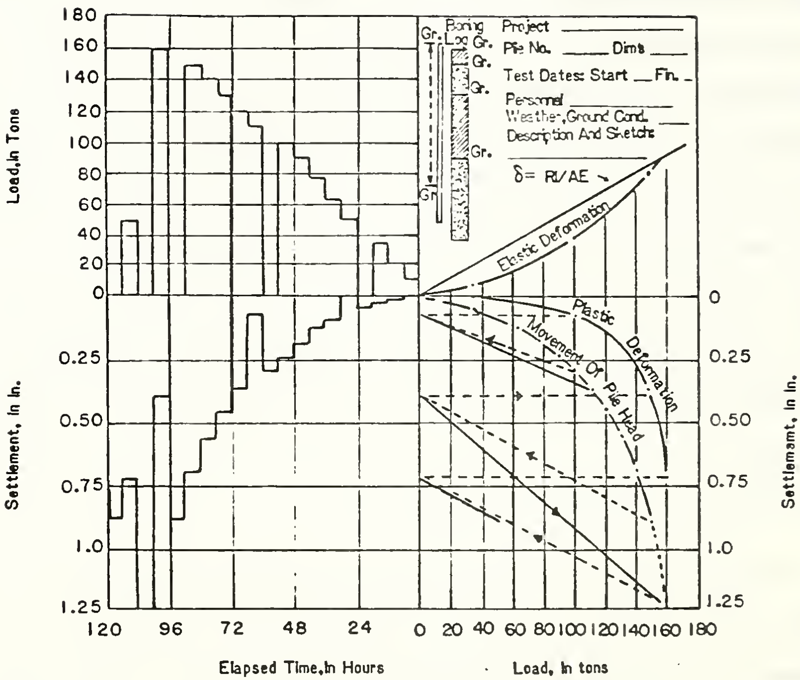


Fig. 17 Test–Loading Diagram Showing Relationship Between Loading, Settlement, And Time. (From Chellis, 1961)

distribution of load between friction and end bearing, using cyclic loading. (This distribution can be determined more accurately using tell-tales as described before.) Van Weele suggested a plotting of the elastic recovery at each unloading cycle versus the load applied at that cycle. The curve usually becomes a straight line soon after the early load increments. The distance between the plotted curve and a line drawn through the origin and parallel to the straight part of the curve represents the portion of the load carried by friction. This is, at best, only an approximation.

Fuller and Hoy (1970) stated that cyclic loading should not be mandatory for routine testing, because it can add unnecessary expense without giving significant additional data. Such special procedures should be included only at the engineer's option.

e. Settlement Behavior:

The load settlement relationship may be used directly to determine the single pile settlement at the working load. It should be noted that the observed settlements made at the top of the pile may not necessarily indicate downward movement of the pile into the ground where high load tests are performed. The possibility of local failure of the pile above ground surface, or crushing of the grout under the test plate, should be recognized as possible factors contributing toward observed settlements.

The complete analysis of the test results should include consideration of all factors, such as the elastic behavior of the pile (from instrumentation or cyclic loading) and an evaluation of the long term performance. This could involve an analysis and evaluation of the subsoil data in conjunction with the test results. In estimating the settlement of a group or the settlement of a pile of different proportions, the average soil modulus (E_s) along the pile may be determined by fitting the measured load settlement behavior to the theoretical behavior. Poulos and Davis (1980) described in detail a theoretical approach to estimate the average drained and undrained values of (E_s) of the clay, using the load test results. Knowing the soil modulus, the stiffness of the pile relative to the soil may be determined, whereby the appropriate theoretical settlement influence factor may be determined for a single pile. Also, the appropriate theoretical group settlement ratio (ratio of group settlement to single pile settlement at the average pile load) may be determined for a pile group. In the latter case, the additional effects of any deep-seated compressible stratum must be carefully considered in the settlement estimate. For the detailed procedures used to determine the above mentioned values, the reader may refer to Poulos and Davis (1980).

f. Some Factors Influencing Interpretation of Test Results:

There are several factors that should be taken into consideration while interpreting the test results. Some of the more important of these are:

1. Changes in pore water pressure in the soil caused by pile driving, construction fill, and other construction operations which may influence the test results for frictional support in relatively impervious soils such as clays and silts.

2. Potential residual loads (discussed earlier) in the pile, which could influence the interpreted distribution of load at the pile tip and along the pile shaft.

3. Possible interaction of friction loads from test piles with upward friction transferred to the soil from anchors or anchor piles (as explained earlier).

4. Differences between conditions at time of installation and after final construction, such as changes in grade or groundwater level.

5. Potential loss of soil supporting the test pile from actions such as excavation and scour.

2.2.2.2 Constant Rate of Penetration Test

This test was developed by Whitaker (1957) for model piles, and it has proved equally useful for full scale pile tests as used by Whitaker and Cooke (1961) and by Whitaker (1963). This test is often called the C.R.P. test. To understand the test, it is helpful to regard the pile as a device for testing the soil, and the pile movement as the means of mobilizing the resisting forces. Thus, Whitaker (1976) defines the ultimate bearing capacity of a pile applicable to the test as "the load at which the full resistance of the soil becomes mobilized".

2.2.2.2.1 Procedure

In carrying out the C.R.P. test, the pile is made to penetrate the soil at a constant speed from its position as installed, while the force applied at the top of the pile to maintain the rate of penetration is continuously measured. The soil supporting the pile is stressed under conditions approaching a constant rate of strain, until it fails in shear, which means that the ultimate bearing capacity of the pile has been reached. The strain rate is adjusted so that the test requires about the same amount of time as an undrained shear test of a sample of the soil in the laboratory. Thus, the two tests have a common basis and connections between them may be legitimately made. The test in this way also provides a speedy way of evaluation and hence is helpful for the overall economy of the project.

Whitaker (1976) emphasized that the purpose of the test is the determination of the ultimate bearing capacity of the pile, and that the force penetration curve obtained in the test does not represent an equilibrium relationship between load and settlement, so that the settlement to be expected under working conditions is not found. Pile movement should be regarded as necessary for mobilizing the forces of resistance.

Since the force applied must be varied smoothly from zero to the ultimate bearing capacity, it is most convenient to use a hydraulic jack. The jack should have a travel greater than the sum of the final penetration of the pile and the upward movement of the reaction system. For an end bearing pile, the penetration in the test may reach 25% of the pile base diameter, while for a friction pile it may reach about 10% of the shaft diameter. According to Whitaker (1976), the movement of the reaction system may be of the order of 76 mm (30.0 in.) if kentledge is used, and about 25 mm (10.0 in.) with a system of anchor piles.

The pump supplying the jack may be hand or mechanically operated. For forces up to about 2000 kN (about 200 metric ton) hand pumping is convenient. However, ASTM D1143-81 recommends the use of a mechanical pump equipped with a bleed valve, variable speed device, or other means for providing a smooth variable delivery. Oil cooling may be required if a bleed value system is used. The jacking force may be measured by a load measuring device, or by a pressure gage in the jack supply line, if the jack is in good condition.

The downward movement of the pile head is conveniently measured by means of a dial gage supported on a beam. According to Whitaker (1976), a small movement of the reference beam (not exceeding 2.5 mm) is not likely to affect significantly the value obtained for the ultimate bearing capacity in a typical C.R.P. test.

The rate of penetration may be controlled by manually checking the time taken for successive small increments of penetration and adjusting the pumping rate accordingly. Alternatively, any mechanical or electrical device may be used to monitor and control the penetration rate so that it remains constant.

A rate of penetration of about 0.75 mm/min (0.03 in./min.) has been found suitable for friction piles in clay, for which the penetration to failure is likely to be less than 25 mm. ASTM D1143-81 suggests a rate of 0.25 to 1.25 mm (0.01 to 0.05 in.)/min for cohesive soils. For end bearing piles in sand or gravel, considerably larger movements will be required to mobilize the full resistance, and rates of penetration of 1.5 mm/min or more are required. ASTM D1143-81 suggests a rate of 0.75 to 2.5 mm (0.03 to 0.10 in.)/min for granular soils. Whitaker (1976) stated that the actual rate of penetration, provided that it is steady, may vary within these specified ranges without significantly affecting the results. For tests performed at rates of penetration varying from 0.018 to 0.128 in. per minute, only 4% variation in the capacity of the piles was observed. A rate should be chosen that can be held by the pumping equipment available. Major fluctuations in the rate of penetration produce corresponding undulations in the force penetration diagram.

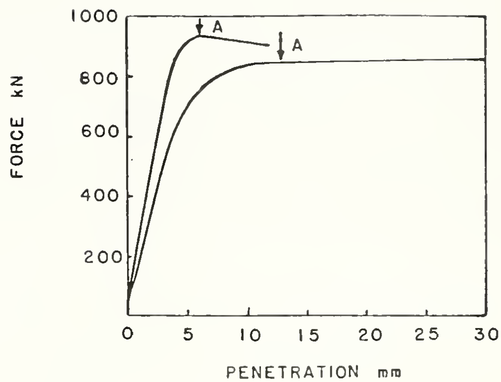
According to ASTM D1143-81, loading the pile should continue until no further increase in the load is necessary for continuous pile penetration at the specified rate. If the pile penetration continues, the load required to achieve the specified penetration rate should be held until the total pile penetration is at least 15% of the average pile diameter or diagonal dimension at which time the load is released. If the pile stops penetrating under the maximum applied load, this load can be released.

Also according to ASTM D1143-81 for the C.R.P. test, readings of time, load and settlement should be taken at least every 30 seconds or at sufficient intervals to determine the rate of penetration being achieved. When the test pile has achieved its specified rate of penetration, readings should be taken for the duration of the loading. Immediately after unloading, readings of time, load and rebound should also be taken. Final readings are taken 1.0 hr after all load has been removed.

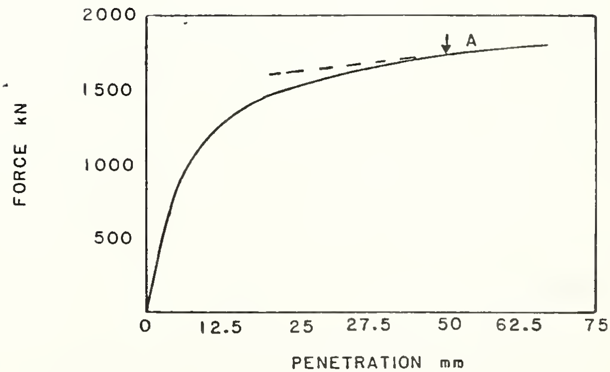
2.2.2.2.2 Interpretation of Test Results:

The data resulting from the test are plotted as a graph of force versus penetration. The curve in the case of a friction pile will be similar to one of those shown in Fig. 18(a). The values of force reached at the point marked "A" would represent the ultimate bearing capacity.

The force penetration curve for an end bearing pile is similar to that shown in Fig. 18(b). The upper part of the curve is



(a)



(b)

Fig. 18 (a)Typical Force—Penetration Diagrams Obtaied With Friction Piles The Values Of Force Reached At Pionts A Represent The Ultimate Bearing Capacity In Each Case. (b) Typical Force—Penetration Diagram Obtained For An End Bearing Pile.
(From Whitaker, 1976)

substantially straight and shows a steady increase in force with increasing penetration. The ultimate load is taken as the point "A", which represents the beginning of the straight portion. This line is found to be a continuation of the force penetration relationship for installation of the pile from the surface of a bearing stratum entirely by a C.R.P. technique (Whitaker, 1976). Identification of the point "A" is often difficult in practice, and Whitaker (1976) suggests that it is usually satisfactory to take the ultimate load as the force required to cause a penetration of 10% of the pile diameter.

Whitaker and Cooke (1961) found that the C.R.P. method provides higher values when compared to maintained load tests. Mohan et al (1967), after performing load tests for the Harduagani Power House in India, reported that the loading points obtained by the C.R.P. method are about 10% higher than the maintained load tests. However, tests performed by the Building Research Station and the British consulting firm of Sir W. Halcrow and Partners, in which the C.R.P. test was compared with the conventional maintained load test procedure, showed that the pile capacities determined by both test procedures were found to be essentially identical for piles driven in soft compressible clay (Fig. 19). For piles driven into granular soils, the C.R.P. tests gave higher ultimate pile capacity than that obtained by maintained load tests (Esrig, 1963). No comparisons was made for piles driven into stiff clay.

Esrig (1963) summarized the advantages of the C.R.P. method over the maintained load method. The major advantage of the C.R.P. method is that it is considerably more rapid and therefore, economical (Fig. (19)). The use of this method on large projects could result in a substantial decrease in the cost of pile tests and in the time required to complete a testing program. Its use on small projects would be particularly advantageous, because the relatively low cost would make pile testing a feasible alternative to the current practice of relying almost exclusively on dynamic pile formulas.

Another advantage of the C.R.P. test is the ease with which the results may be interpreted. The load-deformation curve obtained from the performance of this test shows a definite "break" or change in slope in the vicinity of the maximum load that can be carried by the pile (Fig. 19). This permits the results of the C.R.P. test to be interpreted with greater ease and less reliance on arbitrary definitions of maximum pile capacity than the results of the maintained load tests.

A third, and perhaps minor, advantage of the C.R.P. test is that its use helps to eliminate a possible source of error in interpretation of the results of the pile load tests. This error occurs when the results of a test are used to predict the long term settlements of piles driven into cohesive soils. It is considered likely that the rapidity of the C.R.P. test will suggest to almost everyone that long term settlements cannot always be

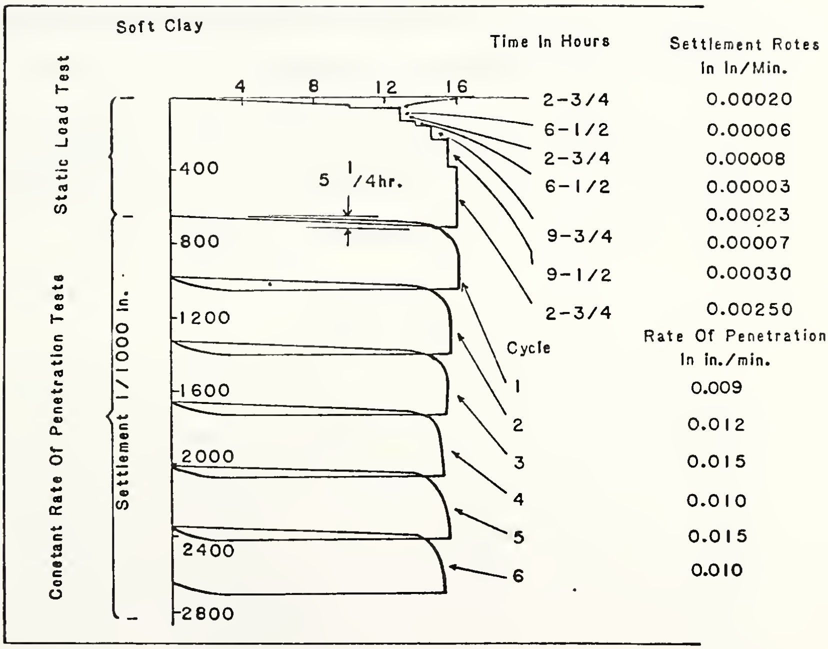


Fig. 19 RESULTS Of Six CRP Tests, Each Made In From 30 To 45 Minutes, Are Similar To Those Of A Static Test In Which The Pile Was Loaded For A Total Of 40 1/2 Hours (From Esrig, 1963)

predicted by means of pile tests. However, it is a disadvantage of the C.R.P. test that it cannot determine the short-term settlement of a single tested pile. Hence, it is not possible to estimate the settlements of pile groups. This is important since the permissible loads on piles bearing in granular material are a function, most frequently, of the predicted settlement of the piles, whereas the permissible loads on piles bearing in cohesive material are related to both the bearing capacity and estimated long term settlement of the piles.

The recent introduction of the C.R.P. method in ASTM standards as an acceptable alternative to the load controlled method will allow its wider application (Vesic, 1977).

2.2.2.3 Method of Equilibrium:

This method was first introduced by Mohan et al (1967). The principal purpose of developing this method was to achieve a state of equilibrium between load and the corresponding settlement. As discussed before, in traditional maintained load testing the applied load is maintained constant either for a fixed period or until the rate of settlement diminishes to a negligible value. This is done to achieve a state of equilibrium between load and settlement. However, this ideal state is seldom reached. Also, with a hydraulic jack, continuous attention and occasional pumping is required to maintain the load, unless a load maintainer is available, and this is both laborious and time consuming.

While carrying out load tests on plates and a number of short bored piles, Mohan and Jain (1964) noticed that equilibrium was reached earlier if a load was first applied which was slightly higher than the desired one. The method has since been studied further and has been found to be very promising. Consequently, the so called "Method of Equilibrium" was introduced by Mohan et al (1967). The main advantages of this method are that it helps to achieve a state of equilibrium between the load and subsequent settlement more quickly, and it eliminates the need of a load maintainer or constant pumping of the jacks to maintain the load at a particular value.

2.2.2.3.1 Procedure:

The principle of this method is to apply to the pile, at each stage of the test, a load slightly higher than the required value, and then to allow the load to decrease itself due to the yielding of the ground, until a state of equilibrium between the load and the settlement exists. By this means, the rate of settlement diminishes much more rapidly than with the maintained load procedure, and equilibrium is reached in a matter of minutes rather than hours. The procedure suggested by Mohan et al (1967) is to first apply one tenth of the estimated ultimate load by hydraulic jack in a period of three to five minutes. This load is maintained for about 5 minutes, and is then allowed to reduce itself via downward movement of the pile. Within a few minutes, a state of equilibrium is generally reached. The next increment of load is then applied and the process is repeated. For higher loads, it is required to maintain the initial load for a period of 10 to 15 min. before it is allowed to relax.

While in sandy soils the state of equilibrium is reached fairly quickly, in clayey soils it takes slightly longer. According to Mohan et al (1967) the total time required for this method is generally reduced to about one third of that required in the maintained load test. At each stage, a cycle of loading and unloading may also be adopted, and the elastic rebound of the pile top measured. This could help in separating skin friction and point bearing capacities (Van Weele, 1957; Jain and Kumar, 1963).

2.2.2.3.2 Discussion and Interpretation of Results:

This procedure was primarily designed to determine the ultimate load capacity, although it also appears to provide reasonable settlement data. From the study carried out by Mohan et al (1967) on long piles, it was evident that the method of equilibrium provides results similar to the maintained load method. An explanation of this behavior can be given by reference to Fig. (20). In a pile load test, it normally takes 3.5 min to apply smaller loads, and 10-15 min to apply higher loads, when the hydraulic jack is operated manually. Two loading cycles for

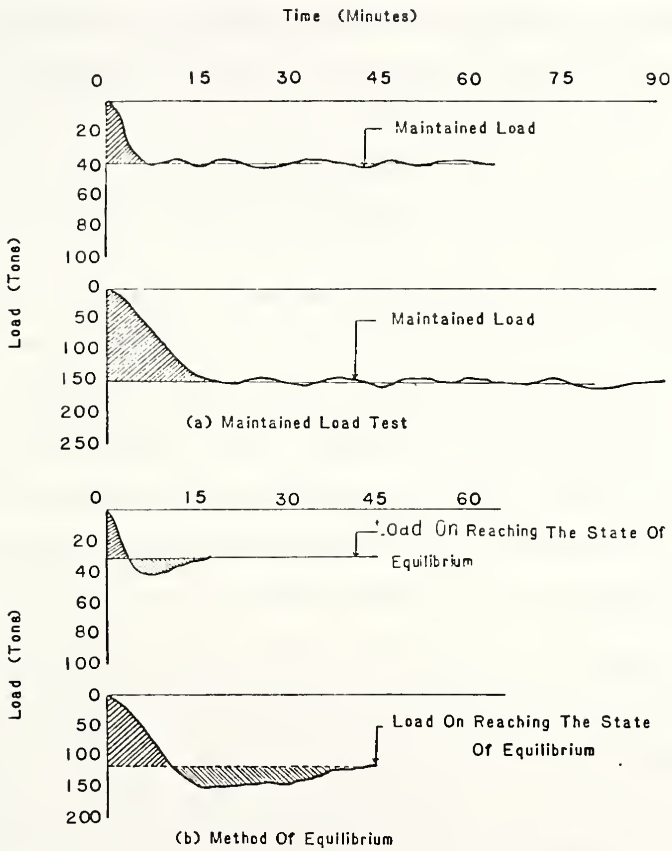


Fig. 20 Load Cycles For Particular Loads.
(From Mohan et al, 1967)

specific loads in a maintained load test are shown in Fig. (20.a). It can be observed that the shaded portions of the curve, which also represent the settlement, are not accounted for. The load which is maintained ultimately, and which is not the average, is taken as the load applied for the period. Another two loading cycles for a test by the method of equilibrium are shown in Fig. (20.b). The load at which equilibrium is attained is always lower than the maximum, so it provides a better average than that obtained in a maintained load test.

Mohan et al (1967) conducted several tests to compare their method with both the maintained load and C.R.P. tests. A series of load tests were performed at seven sites on precast, driven and cast-in-situ piles. A range of pile diameters or sides up to 18 in., was used. Piles penetrated various types of soils, granular and cohesive, of various consistencies. Pile lengths were variable up to a length of 80 ft.

To compare any two tests, the same pile has been used for both tests in order to avoid uncertainties arising from the use of different piles. Unloading was carried out at each stage before application of load by the new method, and one method was immediately followed by the other without allowing any period of rest. This was done purposely to avoid any changes in soil conditions which had been observed earlier to mar the results of other test programs (Mohan et al, 1967).

Among these studies are those shown in Fig. (21) and (22). A typical comparison reveals excellent agreement, in regard to both ultimate load capacity and load settlement behavior, between the method of equilibrium and the maintained load method. The C.R.P. seems to give values about 10% higher than the other two methods (Fig. 22). For more comparison studies refer to Mohan et al (1967).

It then can be concluded that all the interpretations discussed before for the maintained load tests, regarding ultimate and allowable loads and settlements, can be applied to the method of equilibrium.

2.2.2.4 Load Testing by the Texas Highway Department:

Prior to January 1963, the Texas Highway Department (THD) had been using the basic AASHTO 48-24 hour test method as modified by the TDH specifications (1962). Also, Indiana, and surrounding states as well, use the same test. This test is old, complicated and very time consuming, hence it is relatively expensive. Table (2) gives some of the results of several 48-24 hour tests. In some cases, the test duration reached 258 hours (more than 10 days). The average duration was about 126 hours (more than 6 days). Hence, this test is almost never used in other regions. For detailed information about this procedure, the reader may refer to Fuller and Hoy (1970).

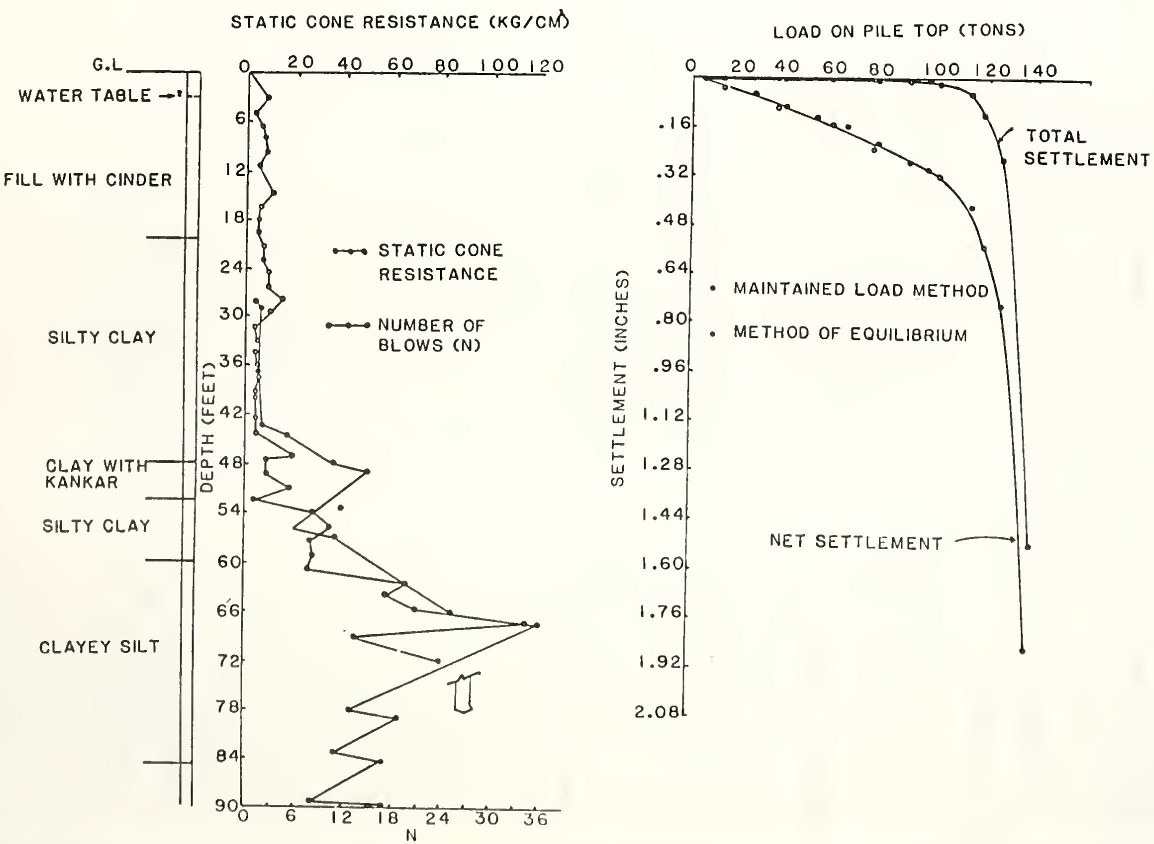


Fig. 21 Load Settlement Curves From Maintained Load Test And Method Of Equilibrium. (From Mohan et al , 1967)

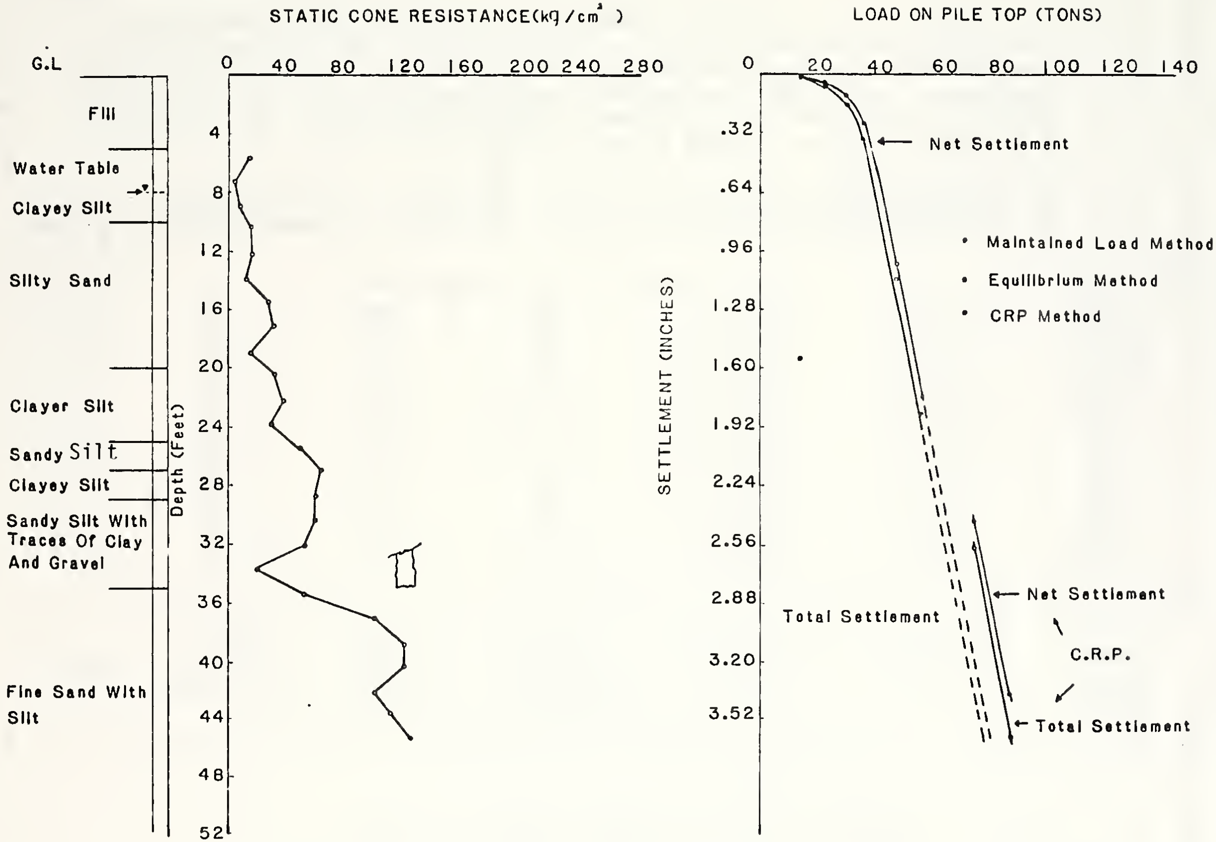


Fig. 22 Load Settlement Curves From Maintainted Load Test, CRP Test And Method Of Equilibrium.
(From Mohan et al, 1967).

Table 2 AASHTO 48-24 Hour Test Method (from Fuller and Hoy, 1970)

Test Number	Duration of Test (hr)	Maximum Load on Pile (tons)	Maximum Load Held 48 Hours (tons)	Gross Settlement (in.)	Net Settlement (in.)	Proven Design Load (tons)	K-Factor[c]
1	102.25	110.0	105.0	0.313	0.251	52.5[a]	1.31
2	55.67	110	110	0.420	0.167	55[b]	1.28
3	57	95	90	0.156	0.049	45[b]	0.54
4	140.25	80	80	0.324	0.161	40[b]	2.67
5	258	155	155	0.379	0.259	77[a]	1.18
6	83.25	75	75	0.412	0.349	35[a]	1.38
7	114.5	120	115	0.562	0.447	56.9[a]	1.46
8	132	160	160	0.501	0.281	79[a]	0.99
9	192	115	115	0.496	0.362	57.52[a]	0.59
10	140.25	115	110	0.566	0.448	52.5[a]	4.67
11	111	105	105	0.390	0.262	50[a]	0.784

Note: Piles loaded by hydraulic jack and reaction beam supported by anchor piling. Settlement was obtained by extensometers.

a. In those cases where the standard 48-24 hour test load caused a permanent net settlement of more than 0.25 in. and other criteria were met, then the maximum proven design load is taken to be 50 percent of that load obtained by interpolation from the computed net settlement line value of 0.25 in. This line was obtained by calculations based on actual recorded recovery.

b. Not failed.

c. $K = \frac{\text{Proven Design Load (AASHTO 48-24 Hour Test Method)}}{\text{ENR Bearing Value}}$

1 inch = 2.54 cm
1 foot = 0.305 m
1 ton = 907.2 kg

After the development of the C.R.P. method by Whitaker and Cooke (1961), described earlier in this report, the Texas Highway Department performed a slight modification of the method (for simplification) and performed several comparison tests using both the standard AASHTO 48-24 hour and the modified C.R.P. methods. These tests gave very promising results, as will be discussed later in this chapter.

2.2.2.4.1 Procedure:

The C.R.P. test calls for records of time and jacking force to be made at equal intervals of movements of the pile head, with the rate of jacking being adjusted so that readings occur at equal intervals of time. For convenience and simplicity, the C.R.P. test was modified by the TDH to produce the quick test method. It requires that loads be added in increments of 5 or 10 tons, with gross settlement readings, loads, and other data being recorded immediately before and after the application of each increment of load. However, ASTM D1143-81 recommends that the load increments be 10-15% of the proposed design load. Each increment is held for 2.5 minutes and the next increment is then applied. When the load deformation curve obtained from these test data (Fig. (23), (24) and (25)) shows that the pile is definitely being failed (i.e., the load on the pile can be held only by constant pumping of the hydraulic jack and the pile is being driven into the ground), pumping is stopped. Gross deformation readings, loads and other data are recorded immediately

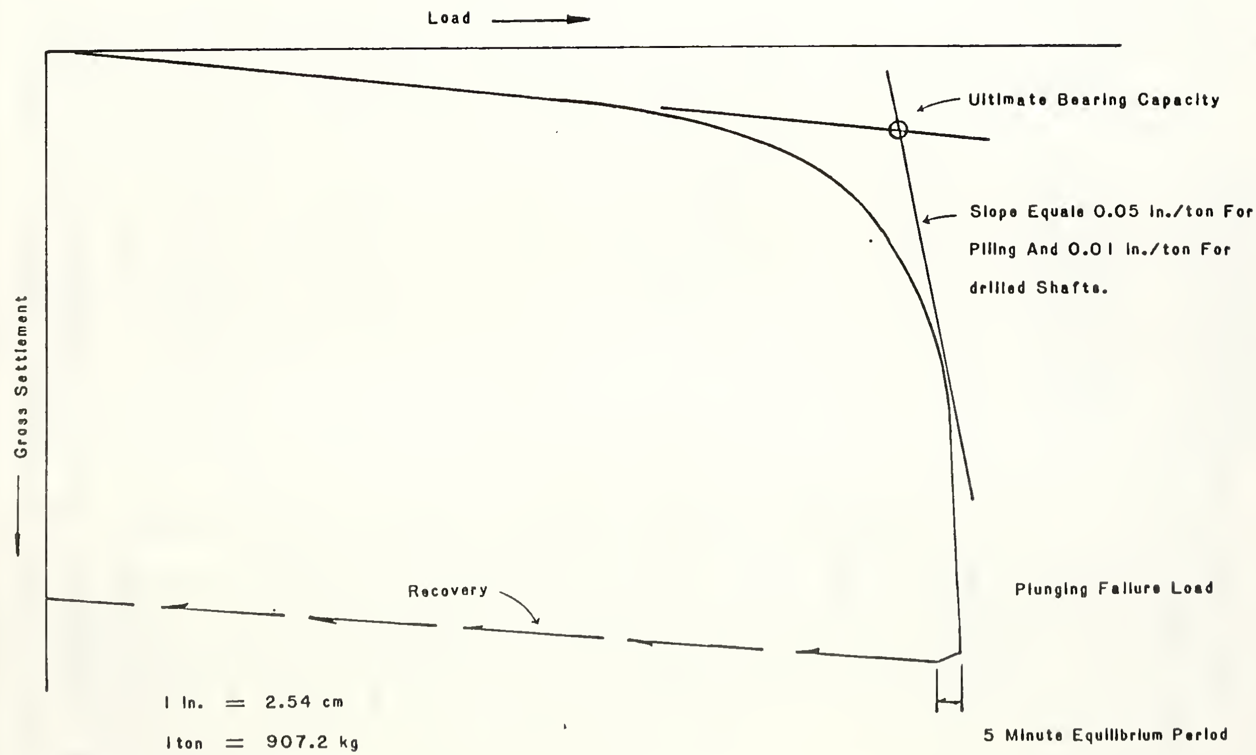


Fig. 23 Typical Load Settlement Graph.(From Butler And Hoy, 1977)

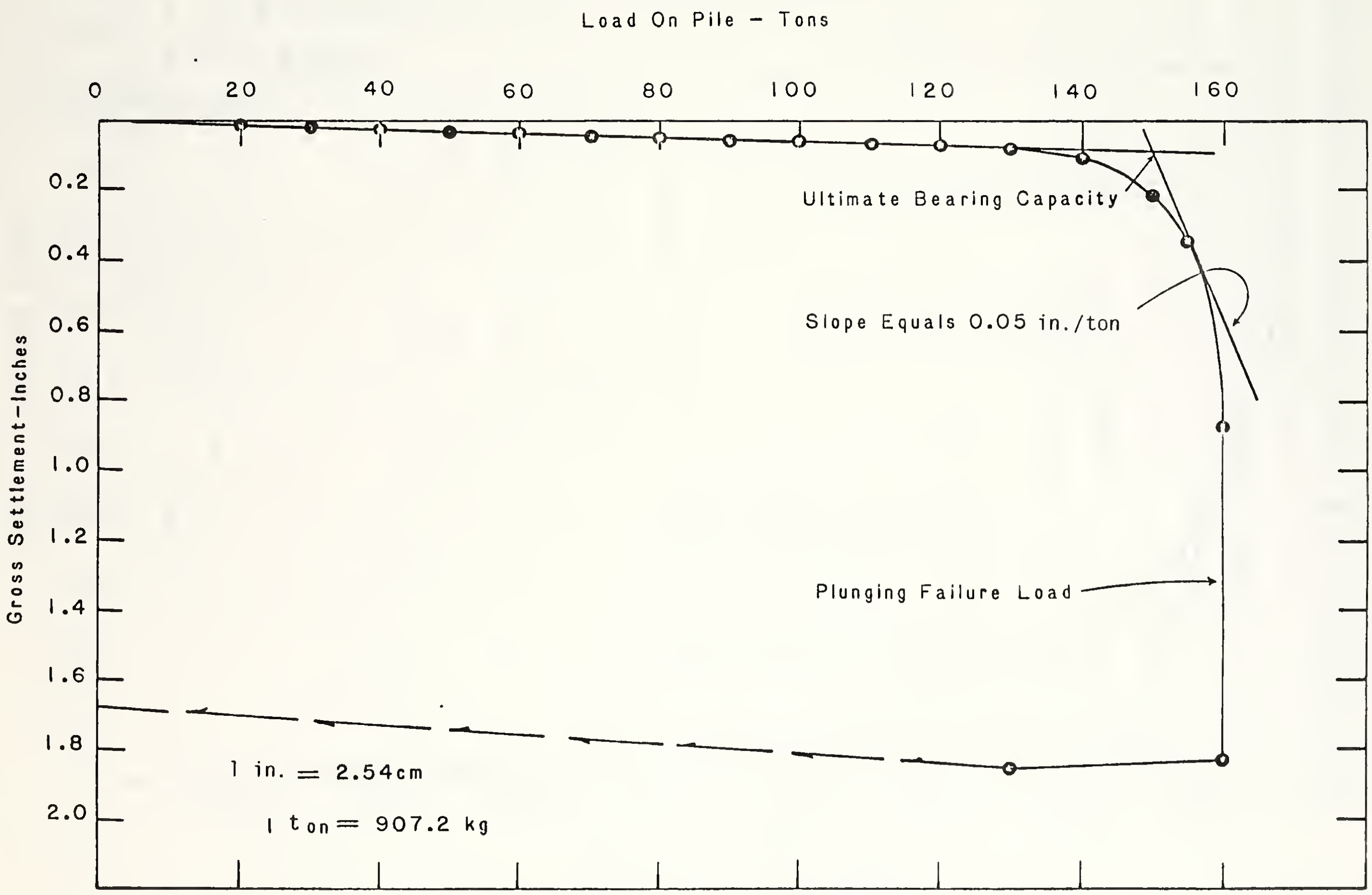


Fig. 24 Load-Settlement Graph For A Pile Load Test

(From Butler And Hoy, 1977)

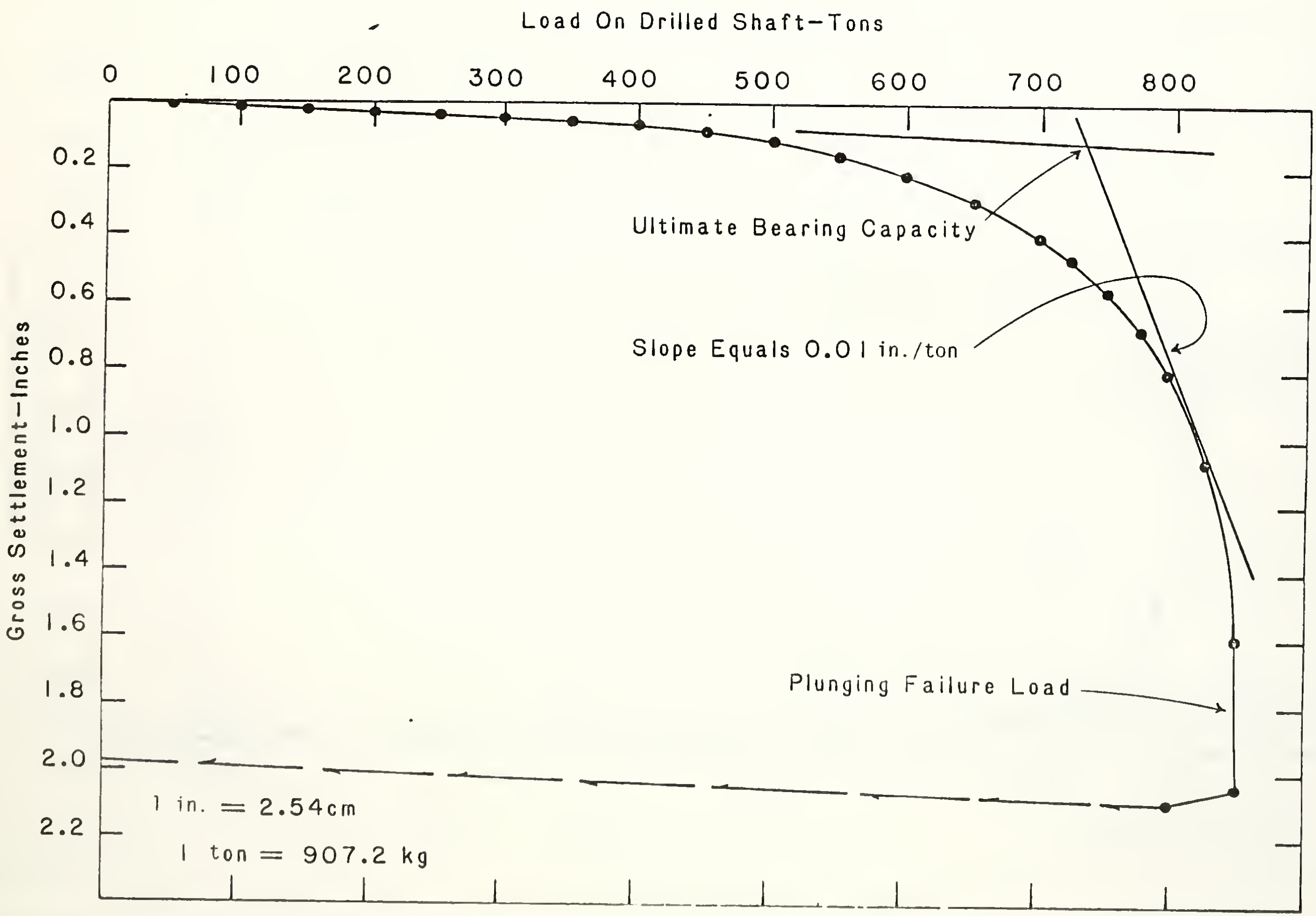


Fig. 25 Load−Settlement Graph For A Drilled Shaft Load Test

after pumping has stopped and again after intervals of 2.5 minutes and 5 minutes. The load on he pile for the case of constant pumping is called the "plunging failure load". Next, all load is removed, and the pile is allowed to recover. However, ASTM D1143-81 recommends that the full test load be removed in four approximately equal decrements, with five minutes between decrements so the shape of the rebound curve may be determined.

All test loads are carried to plunging failure or to the capacity of the equipment. The maximum proven design load is considered to be one half of the ultimate bearing capacity, which is indicated by the intersection of lines drawn tangent to the two basic portions of the load settlement curve as shown in Figs. (23), (24) and (25).

2.2.2.4.2 Correlation Studies by TDH:

The Texas Highway Department conducted 11 pile load tests by using both the standard 48-24 hour and the quick test method. Table 3 summarizes the range used in these tests for piles, hammers and soil types. Table 4 gives the results of the quick test method which can be compared to the results of the standard 48-24 hr test given earlier in Table 2. The maximum proven design load obtained by the quick test method and the 48-24 hour test method are shown in Fig. (26). The average deviation of the maximum proven design load values obtained from the quick test method versus the standard 48-24 hour test method was only about 4% (Fuller and Hoy, 1970).

Table 3 Description of Piles, Soil and Hammer
(from Fuller and Hoy, 1970)

Test Number	Pile Type	Total Pile Length (ft.-in.)	Effective Pile Length (ft.-in.)	General Soil Type	Pile Design Load (tons)	Type of Hammer	ENR Bearing Value (tons)	Final Penetration (in./blow)
1	16 in.sq PC/PS	40 0	36 0	Clay,sand, silty	47	Link-Belt 520	40.2	0.429
2	12BP53	51 0	49 0	Sand,clay	46	McK-T DE-30	42.8	0.260
3	18 in.sq PC/PS	103 0	83 6	Sand,clay	40	Vulcan 014 Vulcan 014	83.7	0.385
4	12BP53	32 4	25 0	Sand,clay	36.4	Vulcan 1	15.0	0.90
5	14 in.sq PC	28 8	21 0	Clay,silty, sandy	53	Vulcan 1	65.2	0.130
6	16 in.pipe	63 8	60 0	-	44	Delmag D-12	25.3	0.444
7	14 by 11 in. step-taper	31 0	31 0	Silt,clay	60	Raymond 1-S	39.0	0.400
8	12BP53	29 0	24 0	Clay,sand, silty	60	Delmag D-12	79.8	0.150
9	12BP53	32 0	31 0	Clay,sand, silty	60	Delmag D-12	97.1	-
10	16 by 11 in. step-taper	47 10	44 0	Clay,silty, sand	31	Raymond 1	11.22	1.20
11	12BP53	21 1	21 0	Sand,clay silty	52	Delmag D-12	63.7	0.170

1 inch = 2.54 cm
1 foot = 0.305 m
1 ton = 907.2 kg

Table 4 Quick Test Method
(from Fuller and Hoy, 1970)

Test Number	Duration of Test (min)	Plunging Failure Load (tons)	Ultimate Bearing Capacity[a] (tons)	Gross Settlement units	Net Settlement units	Proven Design Load (tons)	K-Factor[b]
1	65	120	109	0.185	0.087	54.5	1.36
2	45	145	125	1.151	-	62.5	1.46
3	55	180	150	0.651	0.379	75	0.89
4	65	140	96	0.818	-	48	3.2
5	-	190	166	0.397	0.256	83	1.27
6	50	85	74	0.666	0.596	37	1.46
7	-	134	121.5	0.476	0.356	60.7	1.56
8	75	170	162	0.597	0.371	81	1.01
9	-	120	113.5	0.301	0.168	56.75	0.58
10	105	120	109	0.403	0.284	54.5	4.86
11	67	115	103	0.337	0.226	51.5	0.81

1 inch = 2.54 cm
1 foot = 0.305 m
1 ton = 907.2 kg

a. obtained by Double Tangent Method

b. $K = \dfrac{\text{Proven Design Load (Quick Test Method)}}{\text{ENR Bearing Value}}$

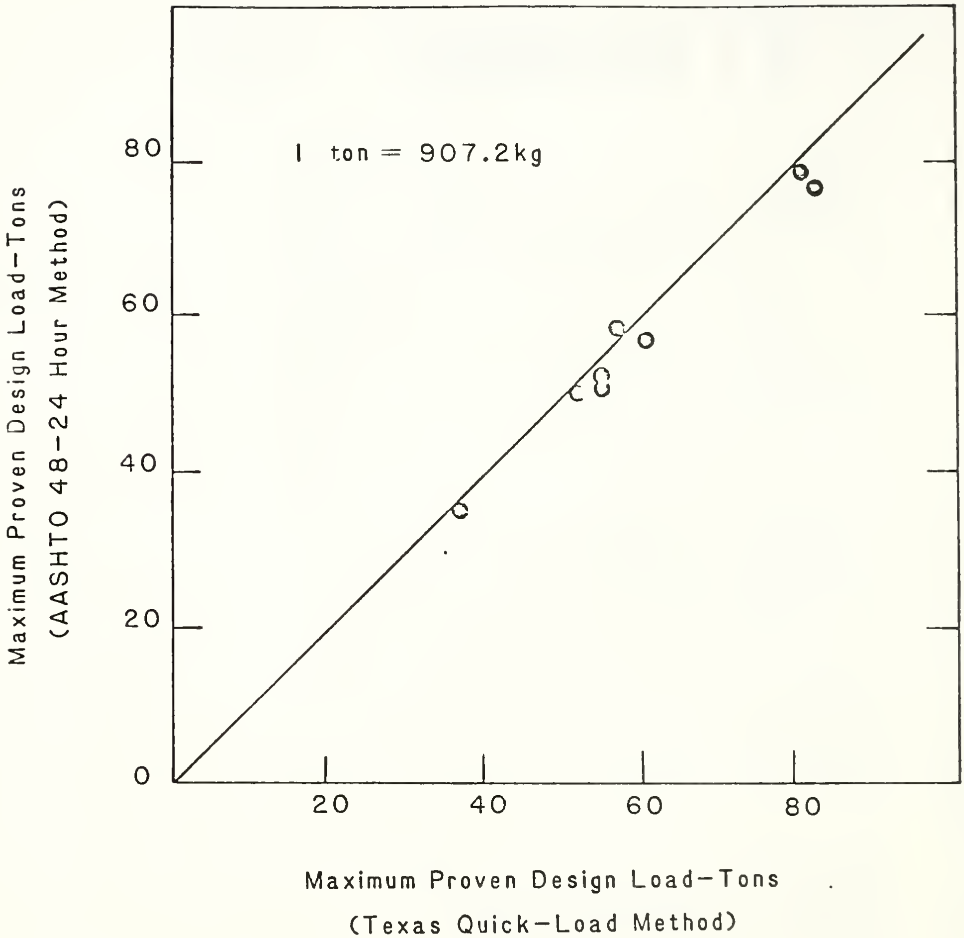


Fig. 26 Correlation Of Proven Design Load Between AASHTO 48-24 Hour And Texas Quick- Load Methods (From Fuller-Hoy, 1970)

The main advantage of the quick method can be clearly shown if the reader refers to Tables 4, 5, 6 and 7. The average total man hours/test for the quick method is only about 1/50 of those required by the 48-24 hour method. Also the time delay to the contractor in the quick method is only about one tenth of that in the other method. This makes full-scale load testing on small projects feasible because of reduced time and cost. In addition, the test is very simple and easy to interpret. The quick tests generally result in more nearly undrained conditions of shear failure, thus providing a condition for which load capacities can be more rationally correlated to static analyses utilizing undrained laboratory shear tests.

According to Butler and Hoy (1977), there are no limitations on the use of the quick load method when it is used to determine the load carrying capacity of a particular foundation soil system. It can not be used, however, to determine the settlement behavior of a foundation under a sustained load. In the case which the settlements need to be defined, i.e. in cases where piles rest on granular soils, an alternative method should be utilized.

Table 5 Average Manpower Requirements and Time Delay to Contractor (from Butler and Hoy, 1977)

Test Method	Average Time/Test	Required Personnel	Man hours/ Test	Time Delay to Contractor
Quick-Load	1.1 hrs.	5	5.5	0.5 days
AASHTO 48-24 Hour	125 hrs.	2	250	5.2 days

Table 6 Estimated Materials Cost for Quick-Load Test (from Butler and Hoy, 1977)

Structural Steel*	$ 0.65/lb.
Dial Indicators (2 required)	130.00
Hydraulic Jack	1,800.00
Miscellaneous (support beams, stakes, etc.)	100.00

*Quantity of structural steel is dependent upon design load.

Table 7 Unit Bid Price for Pile Load Tests in Texas, 1963 through 1975 (from Butler and Hoy, 1977)

Year	No. of Load Tests	No. of Projects	Unit Bid Price
1963	4	3	$1,850
1964	6	3	5,166
1965*	7	3	1,814
1966	6	3	1,400
1967	2	1	4,400
1968	7	4	1,671
1969	11	6	2,681
1970	3	2	5,000
1971	5	3	8,220
1972	2	1	2,500
1973	1	1	4,000
1974	0	0	0
1975	1	1	4,000

*Adopted Quick-Load Method as standard in March of 1965.

the pile is adequate to support the design load (Lambe and Whitman, 1979). Poulos and Davis (1980) cited four reasons for carrying out routine load tests:

1. To serve as a proof test to ensure that failure does not occur before a selected proof load is reached, this proof load being the minimum required factor of safety times the working load.

2. To determine the ultimate bearing capacity as a check on the value calculated from dynamic or static approaches, or to obtain backfigured soil data that will enable other piles to be designed.

3. To determine the load settlement behavior of a pile, especially in the region of the anticipated working load. These data can be used to predict group settlements and settlements of other groups.

4. To indicate the structural soundness of the pile.

According to Fuller and Hoy (1970), the decision to perform the second category of tests, i.e. an advanced test program to develop design criteria, is usually made jointly by the owner and the foundation engineer. This decision is based on the scope of the project and the complexities of the foundation conditions.

2.3 Other Types of Pile Load Tests

2.3.1 Uplift Tests:

Uplift or tension tests on piles can be made at either a continuous rate of uplift (CRU), or an incremental loading basis (maintained load).

A loading rig for an uplift test is shown in Fig. (27). This utilizes nearly the same components as the compressive load testing rig shown in Fig. (2). The methods used for measuring the jacking forces and the movement of the pile head are the same as those used for compressive tests. It is particularly important to space the ground beams or bearers at an ample distance from the test pile. If they are too close, the lateral pressure on the pile induced by the load on the ground surface will increase the skin friction on the pile shaft. Tomlinson (1977) suggested a minimum distance of 2.0 m (6.5 ft), while ASTM 1143-D75 requires a minimum spacing of 2.40 m (8.0 ft).

Where uplift loads are cyclic in character, as in wave loading on a marine structure, it is good practice to adopt repetitive loading on the test pile (Tomlinson, 1977). The desirable maximum load for repeated application cannot be readily determined in advance of the load testing program, since the relationship between the ultimate load for a single application and that for repeated application is not known. Tomlinson (1977) suggested that a single pile should be subjected to a CRU test to

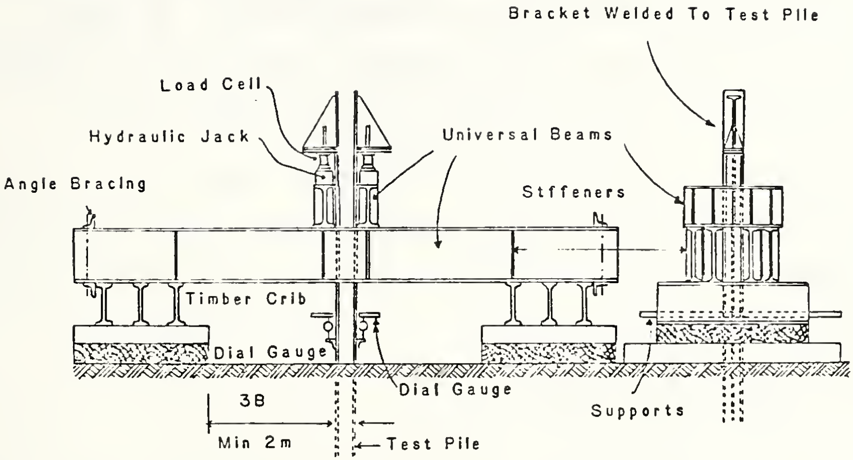


Fig. 27 Testing Rig For Uplift Test On H-Section Pile Using Ground As Reaction.

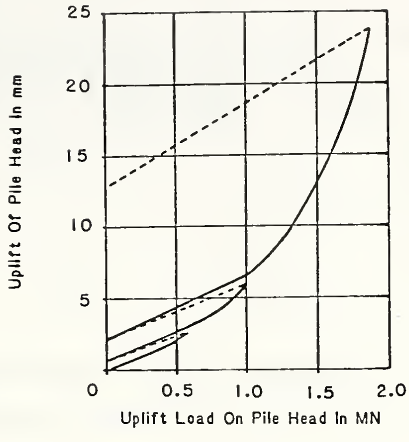
(From Tomlinson, 1977)

obtain the ultimate load for a single application. Next, two more piles should be tested; one cycled at an uplift load of, say, 50% of the single-application ultimate load, and the second at 75% of this value. At least twenty-five load repetitions should be applied. If the uplift continues to increase at an increasing rate after each repetition, the cycling should be continued without increasing the load until failure in uplift occurs. Alternatively, an incremental uplift test can be made with, e.g., ten repetitions of the load at each increment.

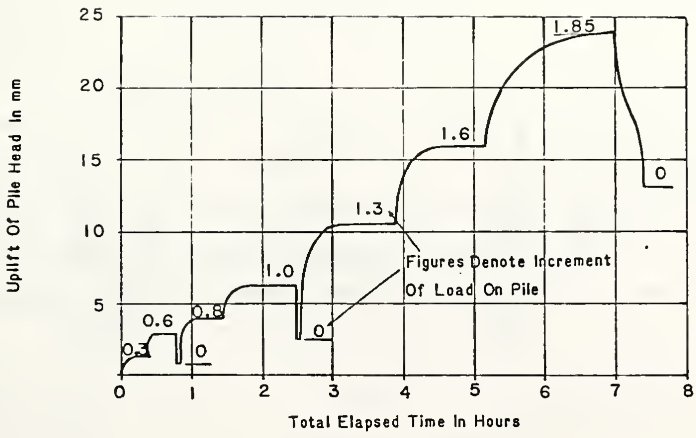
A typical load-time uplift curve for a maintained load test is shown in Fig. (28) (after Tomlinson, 1977). The criteria for evaluating the failure load are similar to those described for compression load tests.

2.3.2 <u>Lateral Load Tests</u>:

Lateral loading tests are usually made by installing a pair of piles and jacking their heads apart. The spacings between the two piles should be sufficient so as not to obtain significant interaction between the movements of each pile, and hence a horizontal beam or tie rods is frequently inserted between the piles. The jack reacts against one of the pile heads and the beam or tie rods to the other pile head (Fig. (29)). A detailed arrangement is shown in Fig. (30) (after Davisson and Stalley, 1970). If necessary, the effect of interaction with less than ideal spacing between the movements of each pile can be



(a)



(b)

Fig. 28 Uplift Load On Test Pile (ML Test) (a) Load–Uplift Curve (b) Time–Uplift Curve (From Tomlinson, 1977)

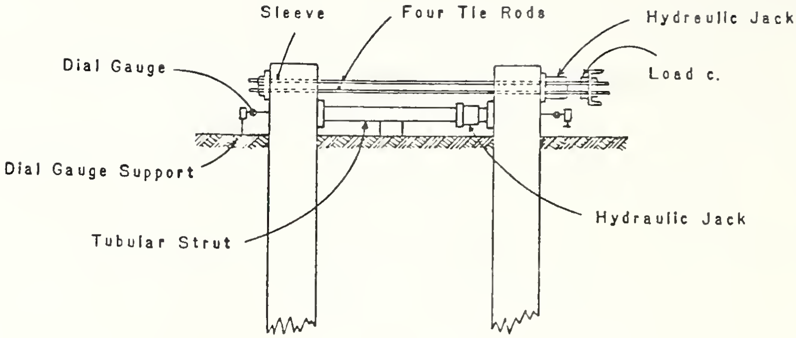


Fig. 29 Testing Rig For Push And Pull Lateral Loading Test On Pair Of Piles(From Tomlinson, 1977)

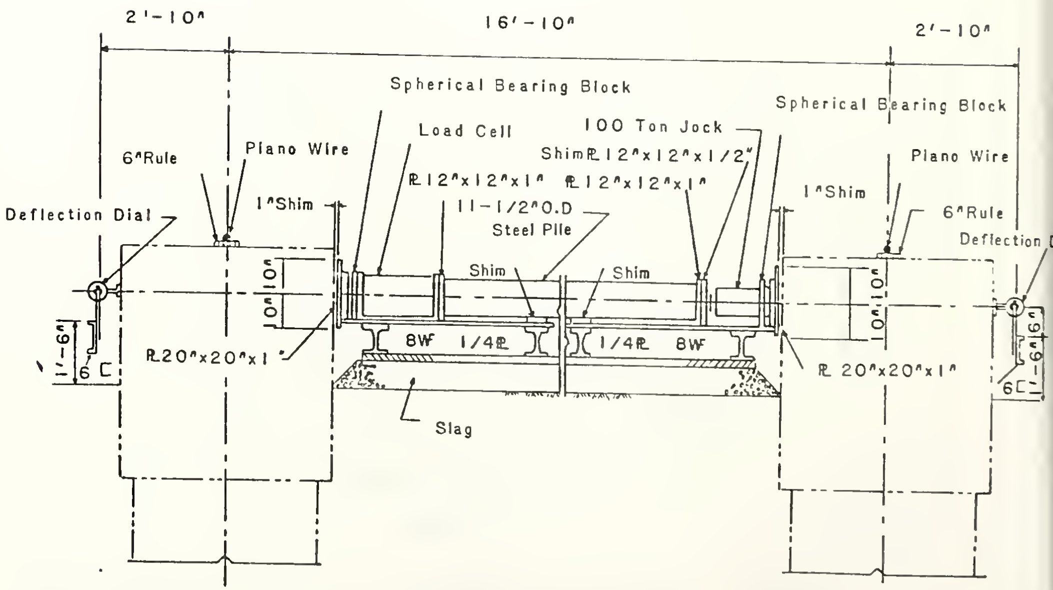


Fig. 30 Typical Setup For Lateral Load Tests. (From Davisson And Stalley, 1970).

estimated, as explained for vertical tests in Section 2.1.5, using theories of pile group-soil interactions for laterally loaded piles (Poulos and Davis, 1980).

The procedure employed for the test varies, but typically the load is applied in a number of increments, and each increment is left on until a specified rate of movement is reached. Alizadeh and Davisson (1970) used, for each increment, a minimum period of one hour, or until the pile head movement was less than 0.01 in. per hour. Tests were carried to lateral deflections approaching 2 in. Where the lateral loads on piles are of repetitive character, as in wave loading or traffic loads on a bridge, it is desirable to make cyclic loading tests (Tomlinson, 1977). This involves alternately pushing or pulling of a pair of piles, using a rig of the type shown in Fig. (29). Instead of a pair of piles, a single pile can be pushed or pulled against a thrust block (Fig. 31).

Diaz et al (1984) performed lateral load tests on prestressed concrete piles penetrating through rockfill in The Port of Los Angeles. In this study, a separate reaction pile was driven behind each "production"/test pile. Each load increment was 8.0 ton (71 kN) in magnitude and applied at a rate of approximately 2.0 tons (18 kN) per minute. Each load step was maintained until the pile head movement stabilized. The loading procedure and cyclic load application methods are described in detail by Diaz et al (1984).

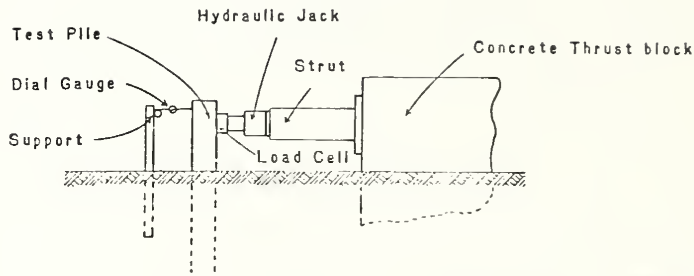


Fig. 31 Test Rig For Lateral Loading Test On Single Pile.(From Tomlinson, 1977)

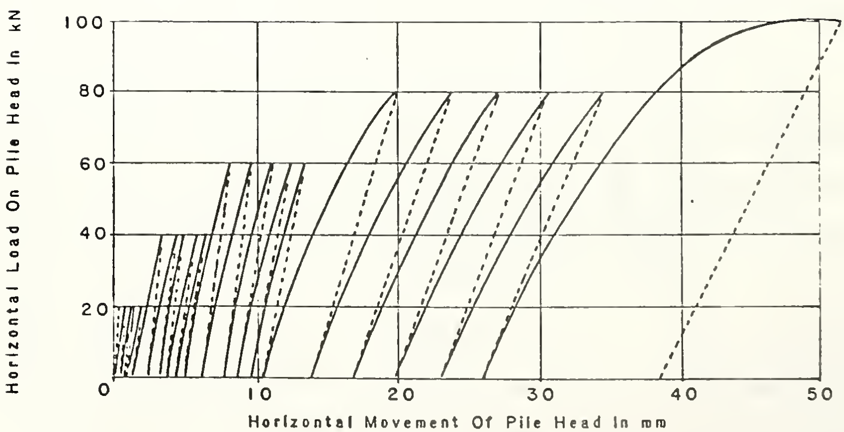


Fig. 32 Load-Deflection Curve For Cyclic Horizontal Loading Test On Pile .(Some Load Cycles Omitted For Clarity).(From Tomlinson, 1977)

Lateral deflection of the pile head is usually measured by dial gages mounted on a frame supported independently of the test piles (Fig. 31). This may not be feasible in marine piles, since the oscillation of the piles and structure supporting the frame in waves and currents may make it impossible to obtain readings with sufficient accuracy (Tomlinson, 1977). Measurements made of the curvature of a pile (the variation of lateral deflection with depth along the pile) by lowering an inclinometer down a tube fixed to the wall of a hollow pile, or cast centrally in a solid pile, are helpful in checking the assumptions made during the theoretical analysis. Hanna (1973) has also employed inclinometer readings along steel H-piles to indicate the bending of piles during driving.

Strain gages are also frequently installed along the embedded portion of the pile to measure flexural stresses, whereby the bending moments may be obtained (Poulos and Davis, 1980). Tomlinson (1977) suggested running two sets of tests when testing piles in marine structures, applying the load at two different elevations, say at the head and just above low water of spring tides. This provides two sets of curves relating deflections to bending moments.

Lateral load tests are usually interpreted to predict the load deflection behavior of a pile. The allowable design load is often taken as the load required to produce a specified deflection (e.g. 0.25 in. according to Poulos and Davis, 1980) divided by the required factor of safety. A typical load deflection curve for cyclic tests are shown in Fig. (32) (from Tomlinson, 1977). The values of the subgrade reaction modulus or the elastic modulus of the soil may also be backfigured from a test, by fitting the observed behavior to the theoretical. This procedure enables predictions to be made of movements of piles of other dimensions or of groups of laterally loaded piles. Poulos and Davis (1980) stated that for laterally loaded pile groups, the influence of any soft layers underlying the pile tips is of much less significance than in the case of vertical loading.

2.3.3 Torsional Testing:

Although axial load tests presently represent the only certain method of determining the ultimate axial load capacity of individual piles, conventional procedures and equipment are relatively costly and inconvenient, especially if high load capacities are anticipated.

An alternative method has been introduced, in which torsional loading tests on piles can be utilized instead of the ordinary axial load tests. Stoll (1972) devised a simple field torque shear load test that could be applied to cylindrical piles. The piles must be capable of carrying the required torque without failure of the pile material itself, so that in stiffer soils, relatively short or stiff piles may be necessary (Poulos and Davis, 1980). The advantage of the test apparatus is that the torque is applied by small capacity hydraulic jacks reacting horizontally against adjoining job piles, utilizing the large mechanical advantage available at the usual spacing (i.e., 3 ft to 5 ft or wider).

According to Stoll (1972), the pile-soil shear strength from torsional tests would not exceed the value for axial loading. Model tests reported by Poulos (1975) showed that in clay, the values deduced from axial and torsional tests were very similar. It was also possible in these tests to backfigure the shear modulus of the soil from the measured torque rotation relationship, using elastic theory, and to use this value to predict the settlement of an axially loaded pile. Thus, there is some evidence that torsional load tests may be useful for predicting the behavior of axially loaded piles in clay. For sands, however, because of the dependence of the pile-soil shear strength on the stress state, torsional tests may give misleading results. Broms and Silberman (1964) obtained, for model piles in sand, considerably lower values from torsional tests than from axial tests.

2.4 Recommended Procedures for Axial Pile Load Tests

2.4.1 General:

In the previous sections of the report, the importance of performing pile load tests to obtain the most valuable and accurate data for design of the pile foundations was emphasized.

In order to make the pile load testing a matter of routine procedure in general practice, costs should be minimized, without compromising the data obtained. To achieve both goals, and based on the information presented in the previous sections of the report, it is recommended that the IDOH uses either the quick load test method or the method of equilibrium for axial pile load tests, depending on the pile-soil system. For friction piles or for piles supported by clayey layers, where settlement data are not to be obtained from the load test, the quick test method should be used to obtain the ultimate pile load capacity. For end bearing piles resting on sand, where settlement information is necessary for the design, the method of equilibrium should be utilized. In cases where the pile is supported by both friction and end-bearing with nearly equal magnitudes, the quick test method can be used for proof testing and in case no settlement data are required. If allowable settlement is the main design criterion, the method of equilibrium should be used.

For both methods, the following general points should be taken into consideration:

1. For driven piles, the test pile should be of the same type and cross section as the piling that will be used in the foundation and should be driven with the same equipment that will be used to drive the foundation piling. A minimum time interval between driving and loading the test pile should be provided. A minimum waiting period of ten days between driving and loading a test pile is recommended. This permits the soil disturbed during the driving operation to regain most of its natural characteristics before testing, without causing an excessive delay to the contractor. Local driving conditions may, however, warrant a different waiting period.

2. For drilled shafts, the test shaft that is to be tested should be constructed using the same method that will be used throughout the structure, i.e., dry hole, cased or slurry displacement. Before loading a test shaft, the concrete must have reached its design strength and the surrounding soil should be given sufficient time to adjust to the changes brought about by the migration of water and cement particles from the fresh concrete (Chuang and Reese (1969)). If the surrounding soil is porous, a layer of soil-cement will form at the concrete-soil interface, resulting in a gain in the soil shear strength. For soils such as

clay, there will be only water migration, and this will cause a decrease in shear strength. Several specifications require a minimum waiting period of seven days between concrete placement and load testing. In any case, testing should not begin before the concrete has attained its design strength.

3. The reaction system for load testing of a pile or drilled shaft must be sufficient to withstand the load required for a plunging failure. Butler and Hoy (1977) suggests that this load be assumed as four times that used in the structural foundation design.

4. Anchorage piles for the reaction system should be spaced to provide a center to center distance from the supports to the test pile of not less than 7 feet (2 m). Closer spacing may be allowed if anchorage piles are to be used as permanent piling for the structure. Using permanent piles as anchorages is preferred whenever possible, as this will reduce the cost of the load test. When this is done, these piles must be checked against uplift forces created by the test load. The effect of these piles on the settlement of the test pile should be investigated.

5. Close attention must be given to the design and fabrication details of the reaction beam because failure

under the high loads attained during a load test would be extremely hazardous. According to Butler and Hoy (1977), a beam that has a working range of 15 to 30 feet (4.6-9.2 m) should be adequate for normal usage.

In addition to the above mentioned points, attention should be given to all the considerations discussed in the preceding sections with respect to loading systems, instrumentation and settlement measurements.

2.4.2 <u>Texas Quick Load Test Method</u>:

As discussed before, the use of this method could result in a tremendous reduction in load test costs. The method is limited to the determination of the pile load capacity, without providing information about the settlement. Hence, it can be used only for cases where settlement data can not be obtained from load tests (e.g., for piles resting on clayey soils). The procedure can be summarized by the following steps:

1. The load is applied in increments of 10 to 15% of the proposed <u>design load</u> with a constant time interval between increments of 2.5 min. The corresponding penetrations are measured and plotted.

2. Load increments are added until continuous jacking is required to maintain the test load (plunging failure),

or until the specified capacity of the loading apparatus is reached, whichever occurs first, at which time jacking is stopped. The load increments should be decreased to a minimum as plunging failure approaches. This procedure will help to define the ultimate load and point of failure more closely.

3. The following data are recorded immediately preceding and immediately following the application of an increment of load: actual time, time interval, load added, total load, dial indicator readings, and total gross penetration. A form similar to that presented on Table 8 is used for recording the data.

4. Load increments are added until plunging failure occurs (whenever continuous pumping of the hydraulic jack is required to maintain load or when the settlement becomes disproportionate to the load being applied). When this occurs pumping should be stopped and data readings are immediately taken. Without pumping, the load and settlement should be allowed to stabilize, making data readings at 2.5 and 5 minutes after pumping is stopped.

5. Quickly and smoothly all load is removed and immediately data readings are taken. Two additional sets of data readings should be taken 2.5 and 5 minutes after

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Table 8

RECORD OF FOUNDATION TEST LOAD

Loading No. ____

TEXAS QUICK TEST LOAD METHOD
(from Butler and Hoy, 1977)

County ____________ Control ____________ Structure ____________

Highway No. ____________ Project ____________ Structure No. ____________

Bent No. ________ Foundation No. ____________ Sta. ____________ Rt. ______ Lt. ______

Foundation Size & Type ____________ Total Length ____________ Design Load ____________

Foundation Tip Elevation ____________ Effective Penetration ____________ Ground Elevation ____________

Hammer Type & Size ____________ Dynamic Resistance ____________

Time Test Began ____________ Date ____________ Resident Engineer ____________

Time	Time Interval	Load Added	Total Load	Extensometer Readings		Total Gross Penetration - Inches		
Min.	Min.	Tons	Tons	Dial 1	Dial 2	Dial 1	Dial 2	Average

Remarks: ____________ District ____________

Date ____________

By ____________

load removal. Tables 9 and 10 are examples of data obtained from axial load tests on driven piles and drilled shafts, respectively.

6. A plot of load versus gross penetration after each 2.5 minutes should be made while the test is being conducted so that its progress can be followed at all times. Hence, it will be possible to recognize the approach to plunging failure and the load increment is then reduced as mentioned before.

7. For the interpretation of load test results, the "double tangent" method is recommended. This is done by the following procedure (refer to Fig. 23).

- Plot a graph of load versus gross penetration using any convenient scale.

- Draw one line originating at the point of zero load and penetration and tangent to the initial flat portion of the gross penetration curve.

- Draw a second line tangent to the steep portion of the gross penetration curve.

- The load at the intersection of the two tangents is defined as the ultimate bearing capacity of the pile or drilled shaft and will be used to establish a proven "maximum safe static" load. This proven safe load for piling is defined

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Table 9

RECORD OF FOUNDATION TEST LOAD

Loading No. 7 Day

TEXAS QUICK TEST LOAD METHOD
(from Butler and Hoy, 1977)

County Gale Control 376-3-48 Structure Intracoastal Canal Bridge
Highway No. State 124 Project BRF 729(6) Structure No. ____

Bent No. 29 Foundation No. 1 Sta. 187+84 Rt. 9' Lt. ____
Foundation Size & Type 18" Sq. Prestr. Conc. Total Length 65' Design Load 60 Tons
Foundation Tip Elevation -59' Effective Penetration 59' Ground Elevation 0.00
Hammer Type & Size Delmag D46-02 Dynamic Resistance 28.86 Tons
Time Test Began 11:00 A.M. Date Aug. 6, 1976 Resident Engineer J. W. Hunter

Time	Time Interval	Load Added	Total Load	Extensometer Readings		Total Gross Penetration - Inches		
Min.	Min.	Tons	Tons	Dial 1	Dial 2	Dial 1	Dial 2	Average
0	0	0	0	2.000	2.000	0	0	0.0000
0	0	10	10	1.998	1.998	.002	.002	0.0020
2.5	2.5		10	1.997	1.998	.003	.002	0.0025
	0	10	20	1.991	1.994	.009	.006	0.0075
5.0	2.5		20	1.991	1.995	.009	.005	0.0070
	0	10	30	1.985	1.990	.015	.010	0.0125
7.5	2.5		30	1.985	1.991	.015	.009	0.0120
	0	10	40	1.978	1.986	.022	.014	0.0180
10.0	2.5		40	1.978	1.986	.022	.014	0.0180
	0	10	50	1.971	1.980	.029	.020	0.0245
12.5	2.5		50	1.970	1.980	.030	.020	0.0250
	0	10	60	1.963	1.975	.037	.025	0.0310
15.0	2.5		60	1.962	1.975	.038	.025	0.0315
	0	10	70	1.956	1.968	.044	.032	0.0380
17.5	2.5		70	1.955	1.968	.045	.032	0.0385
	0	10	80	1.948	1.962	.052	.038	0.0450
20.0	2.5		80	1.947	1.962	.053	.038	0.0455
	0	10	90	1.941	1.957	.059	.043	0.0510
22.5	2.5		90	1.938	1.953	.062	.047	0.0545
	0	10	100	1.932	1.948	.068	.052	0.0600
25.0	2.5		100	1.929	1.944	.071	.056	0.0635
	0	10	110	1.923	1.938	.077	.062	0.0695
27.5	2.5		110	1.920	1.936	.080	.064	0.0720
	0	10	120	1.912	1.928	.088	.072	0.0800
30.0	2.5		120	1.908	1.924	.092	.076	0.0840
	0	10	130	1.900	1.919	.100	.081	0.0905
32.5	2.5		130	1.895	1.915	.105	.085	0.0950
	0	10	140	1.888	1.908	.112	.092	0.1020
35.0	2.5		140	1.882	1.897	.118	.103	0.1105

Remarks: ____ District ____
Date ____
By ____

STATE DEPARTMENT OF HIGHWAYS
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Table 9 (Cont'd)

RECORD OF FOUNDATION TEST LOAD

Loading No. ____

TEXAS QUICK TEST LOAD METHOD
(from Butler and Hoy, 1977)

County ________ Control ________ Structure ________
Highway No. ________ Project ________ Structure No. ________
Bent No. ________ Foundation No. ________ Sta. ________ Rt. ________ Lt. ________
Foundation Size & Type ________ Total Length ________ Design Load ________
Foundation Tip Elevation ________ Effective Penetration ________ Ground Elevation ________
Hammer Type & Size ________ Dynamic Resistance ________
Time Test Began ________ Date ________ Resident Engineer ________

Time	Time Inter-val	Load Added	Total Load	Extensometer Readings		Total Gross Penetration - Inches		
Min.	Min.	Tons	Tons	Dial 1	Dial 2	Dial 1	Dial 2	Average
	0	5	145	1.871	1.887	.129	.113	0.1210
37.5	2.5		145	1.857	1.873	.143	.127	0.1350
	0	5	150	1.841	1.857	.159	.143	0.1510
40.0	2.5		150	1.805	1.822	.195	.178	0.1865
	0	5	155	1.800	1.817	.200	.183	0.1915
42.5	2.5		155	1.619	1.660	.381	.340	0.3605
	0	5	160	1.439	1.518	.561	.482	0.5215
45.0	2.5		160	1.128	1.150	.872	.850	0.8610
47.5	2.5		*160	0.153	0.175	1.847	1.825	1.8360
50.0	2.5	-30	130	0.148	0.173	1.852	1.827	1.8395
52.5	2.5		130	0.147	0.167	1.853	1.833	1.8430
	0	-130	0	0.349	0.271	1.651	1.729	1.6900
55.0	2.5		0	0.354	0.279	1.646	1.721	1.6835
57.5	2.5		0	0.359	0.281	1.641	1.719	1.6800

Remarks: *Plunge Failure - would not take additional load. Stopped pumping.

District 21
Date Aug. 6, 1976
By Walter M. Lane

Table 9 (Cont.)

SUMMARY OF DATA

FOUNDATION TEST LOADING

TEXAS QUICK LOAD TEST METHOD

County________________Structure____________________________________

Highway No.______________Control______________Project_____________

Date of Test Load		Aug. 6, 1976
Bent No.		Bent 29
Location (Station)		187+84
Description of Pile		18" Sq. Prestr. Conc.
Total Length		65′
Ground Elevation		0.00′
Btm. of Ftg. Elev.		-0.97′
Pilot Hole Elev.		0.00′
Pile Tip Elev.		-59.00′
Effective Pen.		59′
Soil Type (General)		Silt, Sand & Clay
Design Load per Pile		60 Tons
Type & Size of Hammer		Delmag D46-02
Dynamic Resistance (ENR)		29 Tons
Penetration per Blow		2.0"
Description of Cushion		Cap Block
Type	Green Oak	Pine Plywood
Size	26" ϕ	23" Sq.
Thickness	6"	3"

	7 Day Test
Duration of Quick Test Load	57.5 Min.
Maximum Load on Pile	160 Tons
Gross Penetration	1.843"
Net Penetration	1.680"
Plunging Failure Load	160 Tons
Ultimate Static Bearing Capacity	150 Tons
Maximum Safe Static Load (Proven)	75 Tons
"K" Factor (Proven)	2.6

Remarks

State Department of Highways
and Public Transportation
District __________________
Date_______________________

STATE DEPARTMENT OF HIGHWAYS
AND PUBLIC TRANSPORTATION
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Table 10

RECORD OF FOUNDATION TEST LOAD

Loading No. ____

TEXAS QUICK TEST LOAD METHOD
(from Butler and Hoy, 1977)

County Morton Control 37-13-2 Structure H&T RR Overpass
Highway No. IH 45 Project I 45-1(151)037 Structure No. ____

Bent No. 20 Foundation No. ____ Sta. 137+10 Rt. 10' Lt. ____
Foundation Size & Type 36" Ø Drilled Shaft Total Length 60' Design Load 300 Tons
Foundation Tip Elevation +6' Effective Penetration 50' Ground Elevation +66.2'
Hammer Type & Size ____ Dynamic Resistance ____
Time Test Began 10:00 A.M. Date 8-18-76 Resident Engineer J. B. Thomas

Time	Time Interval	Load Added	Total Load	Extensometer Readings		Total Gross Penetration - Inches		
Min.	Min.	Tons	Tons	Dial 1	Dial 2	Dial 1	Dial 2	Average
0	0	0	0	2.000	2.000	0.000	0.000	0.0000
0	0	50	50	1.995	1.995	0.005	0.005	0.0050
2.5	2.5		50	1.995	1.995	0.005	0.005	0.0050
	0	50	100	1.991	1.989	0.009	0.011	0.0100
5.0	2.5		100	1.990	1.988	0.010	0.012	0.0110
	0	50	150	1.984	1.982	0.016	0.018	0.0170
7.5	2.5		150	1.982	1.980	0.018	0.020	0.0190
	0	50	200	1.977	1.975	0.023	0.025	0.0240
10.0	2.5		200	1.974	1.972	0.026	0.028	0.0270
	0	50	250	1.966	1.964	0.034	0.036	0.0350
12.5	2.5		250	1.963	1.961	0.037	0.039	0.0380
	0	50	300	1.957	1.955	0.043	0.045	0.0440
15.0	2.5		300	1.954	1.952	0.046	0.048	0.0470
	0	50	350	1.946	1.945	0.054	0.055	0.0545
17.5	2.5		350	1.943	1.942	0.057	0.058	0.0575
	0	50	400	1.928	1.927	0.072	0.073	0.0725
20.0	2.5		400	1.924	1.923	0.076	0.077	0.0765
	0	50	450	1.907	1.905	0.093	0.095	0.0940
22.5	2.5		450	1.902	1.900	0.098	0.100	0.0990
	0	50	500	1.880	1.877	0.120	0.123	0.1215
25.0	2.5		500	1.874	1.871	0.126	0.129	0.1275
	0	50	550	1.844	1.841	0.156	0.159	0.1575
27.5	2.5		550	1.837	1.834	0.163	0.166	0.1645
	0	50	600	1.783	1.779	0.217	0.221	0.2190
30.0	2.5		600	1.776	1.772	0.224	0.228	0.2260
	0	50	650	1.721	1.715	0.279	0.285	0.2820
32.5	2.5		650	1.713	1.707	0.287	0.293	0.2900
	0	50	700	1.603	1.597	0.397	0.403	0.4000
35.0	2.5		700	1.593	1.587	0.407	0.413	0.4100

Remarks: ____ District ____
Date ____
By ____

STATE DEPARTMENT OF HIGHWAYS
AND PUBLIC TRANSPORTATION
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Table 10 (Cont'd)

RECORD OF FOUNDATION TEST LOAD

Loading No. ____

TEXAS QUICK TEST LOAD METHOD
(from Butler and Hoy, 1977)

County ____ Control ____ Structure ____
Highway No. ____ Project ____ Structure No. ____

Bent No. ____ Foundation No. ____ Sta. ____ Rt. ____ Lt. ____
Foundation Size & Type ____ Total Length ____ Design Load ____
Foundation Tip Elevation ____ Effective Penetration ____ Ground Elevation ____
Hammer Type & Size ____ Dynamic Resistance ____
Time Test Began ____ Date ____ Resident Engineer ____

Time	Time Inter-val	Load Added	Total Load	Extensometer Readings		Total Gross Penetration - Inches		
Min.	Min.	Tons	Tons	Dial 1	Dial 2	Dial 1	Dial 2	Average
	0	2.5	725	1.544	1.534	0.456	0.466	0.4610
37.5	2.5	0	725	1.534	1.524	0.466	0.476	0.4710
	0	2.5	750	1.451	1.441	0.549	0.559	0.5540
40.0	2.5	0	750	1.441	1.431	0.559	0.569	0.5640
	0	2.5	775	1.334	1.322	0.666	0.678	0.6720
42.5	2.5	0	775	1.325	1.313	0.675	0.687	0.6810
	0	2.5	800	1.194	1.176	0.806	0.824	0.8150
45.0	2.5	0	800	1.179	1.161	0.821	0.839	0.8300
	0	2.5	825	0.977	0.956	1.023	1.044	1.0335
47.5	2.5	0	825	0.958	0.937	1.042	1.063	1.0525
	0	2.5	850	0.710	0.699	1.290	1.301	1.2955
50.0	2.5	0	850	0.410	0.388	1.590	1.612	1.601
*50.0	0	0	850	1.410	1.388	1.590	1.612	1.601
52.5	2.5	0	**850	0.968	0.946	2.032	2.054	2.043
55.0	2.5	-50	800	0.923	0.901	2.077	2.099	2.088
57.5	2.5		800	0.920	0.898	2.080	2.102	2.0910
	0	-800	0	1.008	0.986	1.992	2.014	2.0030
60.0	2.5		0	1.038	1.020	1.962	1.980	1.9710
62.5	2.5		0	1.042	1.034	1.958	1.966	1.9620

Remarks: *Extensometers reset with 1" spacers
immediately after reading.
**Plunge failure load at 850 tons.

District ____
Date 8-18-76
By H. M. Lott

Table 10 (Cont.)

SUMMARY OF DATA

FOUNDATION TEST LOADING

TEXAS QUICK LOAD TEST METHOD

County________________Structure________________________________

Highway No.___________Control______________Project____________

Date of Test Load	8/18/76
Bent No.	20
Location (Station)	137+10 (10´ Rt.)
Description of Shaft	36"0 Drilled Shaft
Total Length	60´
Ground Elevation	+66.2
Shaft Top Elev.	+65.0´
Shaft Tip Elev.	+6.0´
Effective Pen.	50.0´
Soil Type (General)	Clay, Silt, Sand
Method of Installation	Casing w/Slurry
Design Load per Shaft	300 Tons
Allowable Point Bearing Load (Lab. Tests)	73 Tons
Allowable Frictional Load (Lab. Tests)	395 Tons

Duration of Quick Test Load	62.5 Min.
Maximum Load on Shaft	850 Tons
Gross Penetration	2.200"
Net Penetration	1.962"
Plunging Failure Load	850 Tons
Ultimate Static Bearing Capacity	732 Tons
Maximum Safe Static Load (Proven)	366 Tons
"K" Factor (Proven)	0.74

Remarks

Date________________

as one-half of the ultimate bearing capacity obtained from the graph. For the drilled shafts, the same factor of safety is used, in addition the gross penetration for the proposed design load should not be more than 0.5 in. (1.3 cm).

2.4.3 Method of Equilibrium:

The method of equilibrium can be used when settlement data are needed and can be quickly obtained. This is the case for end bearing piles resting on sand, where settlements are needed to calculate the allowable pile load. The principle of this method is to apply to the pile, at each stage of the test, a load slightly higher than the required value and then let it decrease itself to the the desired value. By this means, the rate of settlement diminishes much more rapidly than with the maintained load procedure. Equilibrium is reached in a matter of minutes rather than hours.

The test procedure can be summarized as follows:

1. The test load (ultimate load) is assumed to be twice the proposed design load.

2. About one tenth of the estimated <u>ultimate load</u> is applied by a hydraulic jack in a period of 3-5 minutes.

3. The load increment is maintained for about five minutes, and then allowed to reduce itself due to the yielding of the ground, until a state of equilibrium between the load and settlement exists. This usually takes not more than a few minutes. It takes a slightly longer time for clayey soils than for sandy soils. For pile carrying relatively high loads, it is desirable to maintain the initial load for a period of 10-15 minutes before it is allowed to diminish.

4. The next load increment is then applied and the procedure is repeated.

5. At each stage of loading, a cycle of loading and unloading may also be adopted and the elastic rebound of the pile top measured. This is done only if the net settlement load curve is required.

6. Loading is continued to the proposed ultimate load, provided that the test pile has not failed. The total test load is then left for 12 hours, if the butt settlement over a one-hour period is not greater than 0.01 in. (0.25 mm); otherwise, the load is allowed to remain on the pile for 24 hrs. If pile failure occurs, jacking the pile should continue until the settlement equals 15% of the pile diameter or diagonal dimension.

7. The test load is then removed in decrements of 25% of the total test load.

8. The results of this test may be interpreted exactly like the standard 48-24 hr. method test results.

A data sheet similar to that given in Table 4-1 can also be used to record the results of load tests performed by the method of equilibrium. Examples of the plotted results were given in Fig. (21) and (22).

CHAPTER 3

DYNAMIC MEASUREMENTS FOR PILE DRIVING

3.1 Introduction

The technology of design and construction of pile foundations has been improving dramatically during the last two decades. In spite of the recent developments that resulted in: more accurate methods of predicting pile capacities; improved methods of construction control; and the use of highly specialized methods and equipment for driving, some uncertainties still exist. Thus far, static pile load tests have proven to be the best way to obtain relatively accurate information regarding the static capacity of the pile. On the other hand, the performance of the pile and driving system during driving cannot be monitored successfully without the use of dynamic measurements. These measurements have been used by many investigators and organizations to examine the pile driving procedure (e.g. by Fellenius, 1984; Goble et al., 1970, 1975, 1978; Rausche, 1985). These investigations have shown that the dynamic measurements, together with the wave equation analysis, can be used for many purposes, divided into three main categories. These are the pile capacity, the hammer performance and the pile performance and integrity.

3.1.1 Pile Capacity:

Dynamic measurements can be used in:

- Investigating the pile capacity at time of testing, set-up relaxation or driveability.

- Quality control by restriking several piles within a few hours.

- Determining capacity versus penetration relationship.

- Searching for adequate bearing strata for a given capacity.

- Confirming if current criteria like blow count or penetration are adequate for capacity, or reducing driving time with less risk of pile structural damage with improved criteria.

- Increasing pile allowable load if soil capacity is higher than expected.

- Reducing the number of necessary static load tests for a project.

- Obtaining force - acceleration data, which later can be used to determine static soil resistance distribution and damping parameters for wave equation input, or to estimate expected pile deformation by simulating a static load test.

3.1.2 Hammer Performance

The dynamic measurements can be used to indicate:

- The measured energy actually transmitted to the pile. Comparison with the manufacturer's rated value evaluates the hammer performance.

- Effects of changes in cushion properties or helmet assemblies, by their effect on transmitted energy and forces in the pile.

- The efficiencies of different operating pressures, strokes or batters, or changes in hammer maintenance conditions by comparative testing of hammers of one type, or of a single hammer over an extended period of time.

- Relative efficiencies of different hammer types.

- Preignition problems with diesel hammers.

- Whether soil or hammer is responsible for changes in blow count.

3.1.3 Pile Performance and Integrity:

The dynamic measurements can be used to indicate:

- Magnitude of driving stresses at the same measuring location. Thus, piles can be driven in a way to minimize local buckling or tension cracking.

- The effect on driving stresses due to changes in the driving system.

- The entire stress history in the pile, derived from force and acceleration at the pile top, with further analysis.

- The extent and location of pile structural damage. Thus, costly extraction is not necessary to confirm damage.

- The total length of existing piles where the lengths are unknown.

- The structural modulus of elasticity of concrete piles.

- Most economical pile type for given capacity and safe driving stresses.

3.2 Technical Background on Dynamic Measurements

Developments in electronics during the past 30 years provide a potential for a substantial change in the methods used for design, construction control and analysis of piles. These developments have made possible dynamic measurements of force and acceleration at the pile top.

Dynamic measurements can be used for measuring four major quantities: force, displacement, velocity and acceleration (Goble and Rausche, 1970). Force measurements can be made reliably and accurately with the availability of soil resistance strain gages. Dynamic displacement measurements in the environment common in pile driving are quite difficult. A fixed reference must be maintained for instruments with limited stroke as the pile moves downward. Considerable attention must be given to the attachment of the transducer to the pile to assure that it is sufficiently rigid to transmit the motion of the pile directly to the instrument without substantial modification. Goble and Rausche (1970) recommend the use of two transducers, since the pile at the point of measurement may rotate due to eccentric loading. Velocity measurements are complicated by all of the considerations mentioned above for displacement measurements due to the basic similarity of the transducers. Acceleration measurements are made somewhat easier by the availability of a wide variety of commercial devices. The fact that a reference frame is readily available generally simplifies these measurements.

The above mentioned discussion indicates that it is much easier to measure acceleration than velocity and displacement. Since these two quantities can be obtained by integrating acceleration measurements, emphasis has been given to two measurements only at the pile top, i.e. force and acceleration. In the beginning available instruments were used for that purpose (Goble and Rausche, 1970), but they had to be modified or improved to justify their use on a routine basis.

3.2.1 Acceleration Measurement

Accelerations are measured with an appropriate type of accelerometer attached to the pile a short distance below its top. A large variety of accelerometers are commercially available and probably many of them would be satisfactory in the measurement of acceleration during pile driving. Two general types of accelerometers are readily available commercially (Goble and Rausche, 1970). One of these makes use of a mass mounted on a very stiff spring with strain gages mounted on the spring. The other one is the piezoelectric accelerometer which makes use of the fact that a weak electronic signal is produced by a quartz crystal when it is subjected to an acceleraton. This signal can be calibrated to acceleration. The later type was the one which was used by Goble and his colleagues for taking acceleration measurements.

The most important consideration in the selection of an accelerometer is the frequency response of the device. The nature of typical pile acceleration records requires that the accelerometer should respond to frequencies from at least 1200-1500 hertz down to nearly static (Goble and Rausche, 1970). Again, this requirement is dependent on the use to be made of the measurements. Strain gage type accelerometers do not easily reach the upper end of this requirement but do provide a response down to static. Piezoelectric accelerometers are available with extremely high frequency limits but must be selected carefully on the low end. Studies made at Case Western Reserve (Rausche, 1970) showed that the Kistler Model 818 was the best accelerometer to use since it has a built-in amplifier and thus produces a strong signal. With this transducer, successful acceleration measurements have been made at the tip of pipe piles (Rausche, 1970).

Another source of error in dynamic acceleration measurements is the use of excessively flexible mounts. The natural frequency of the base should be well above the maximum frequency to be recorded.

Goble and Rausche (1970) recommend the use of two accelerometers and the averaging of the two readings, so that the centroidal acceleration can be determined, and any rotational motion eliminated.

3.2.2 Force Measurement

Two methods were used during the "Case" project for force measurement (Goble and Rausche, 1970). The first technique used was the measurement of strain in the piles then with a knowledge of the pile area and the material modulus of elasticity, the force can be determined. A second method was the installation of a force transducer in the driving system.

According to Goble and Rausche (1970), resistance strain gages provided a very precise and convenient strain measurement. However, field installation is tedious, time consuming and requires considerable skill. For this reason Goble et al. (1968) developed a transducer which can be easily attached to the pile and which gives a readout of strain at the point of attachment. A drawing of the design is shown in Fig. (33). The gages are placed at locations of tension and compression stress. A transducer is used on each side of the pile to cancel out the effect

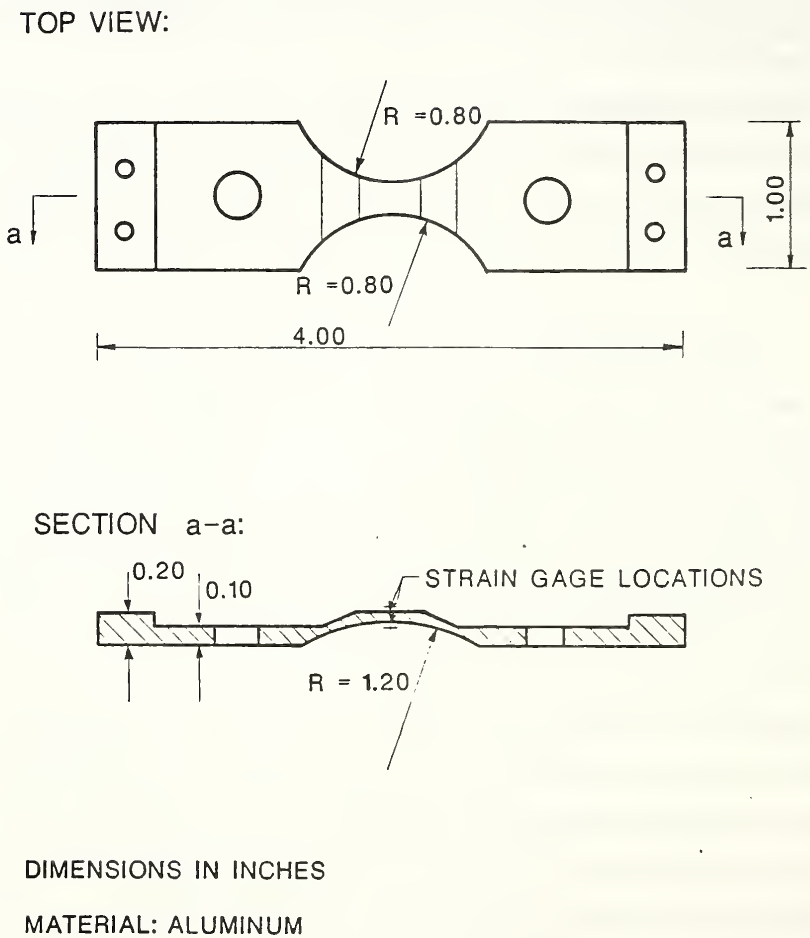


Fig. 33 Strain Transducer Design
(from Goble and Rausche, 1970)

of gross bending. The gages of the two transducers are connected so that those experiencing strains of the same direction are in series. Thus, the two transducers form 240-ohm arms of the bridge. The other two arms are contained in a termination box. The transducers are bolted directly to the pile with self-tapping bolts. They should be calibrated properly before being used. Details of the calibration process were described by Goble and Rausche (1970). The following observations were drawn from experience with the device at Case Western:

1. Local wall bending is not a problem unless extremely short lengths are used, therefore, gages on the outside are unnecessary.

2. As the transducer length is decreased, the number of strain gages must be increased.

3. The latest version of the force transducer minimized any effect on the dynamic behavior, since its elastic and internal properties are close to those of the pile.

3.2.3 Recording Devices

Two types of instruments can be used for recording the measurements, oscilloscopes and oscillographs. The oscilloscope has the advantage of very wide ranges of speed and frequency response. However, for a transient phenomenon it does not display continuously, and to obtain a permanent record the scope display must be photographed. For this reason it cannot be considered for any routine repetitive measurements, but it is useful for checking other recording devices (Goble and Rausche, 1970). On the other hand, oscillographs have been used extensively in pile dynamic measurements. The Visicorder type has been found to be very efficient (Rausche, 1970). Current models of this instrument operate at speeds up to 160 inches per second. For pile dynamic recording, 80 inches per second has been found to be satisfactory, although the additional speed would make record evaluation easier. Also galvanometers provide a nearly linear output up to 8 khz, well above the range required. Acceleration measurements have been satisfactorily recorded using 1500 khz galvanometers, and the force signal is even less demanding. An example of the oscillgraph record is shown in Fig. (34). It is possible that magnetic tape recorders may provide an even more desirable system.

The first step in the examination of all pile dynamic data is to check them. Goble et al., (1970) introduced a simple relation to make this check. It can be shown that there is a linear relationship between force, F(t), and velocity, v(t), in the early part of the record if both are measured at the same location. This relation is given by:

$$F(t) = \frac{EA}{c} \cdot v(t) \tag{3.1}$$

where: E = material modulus of elasticity

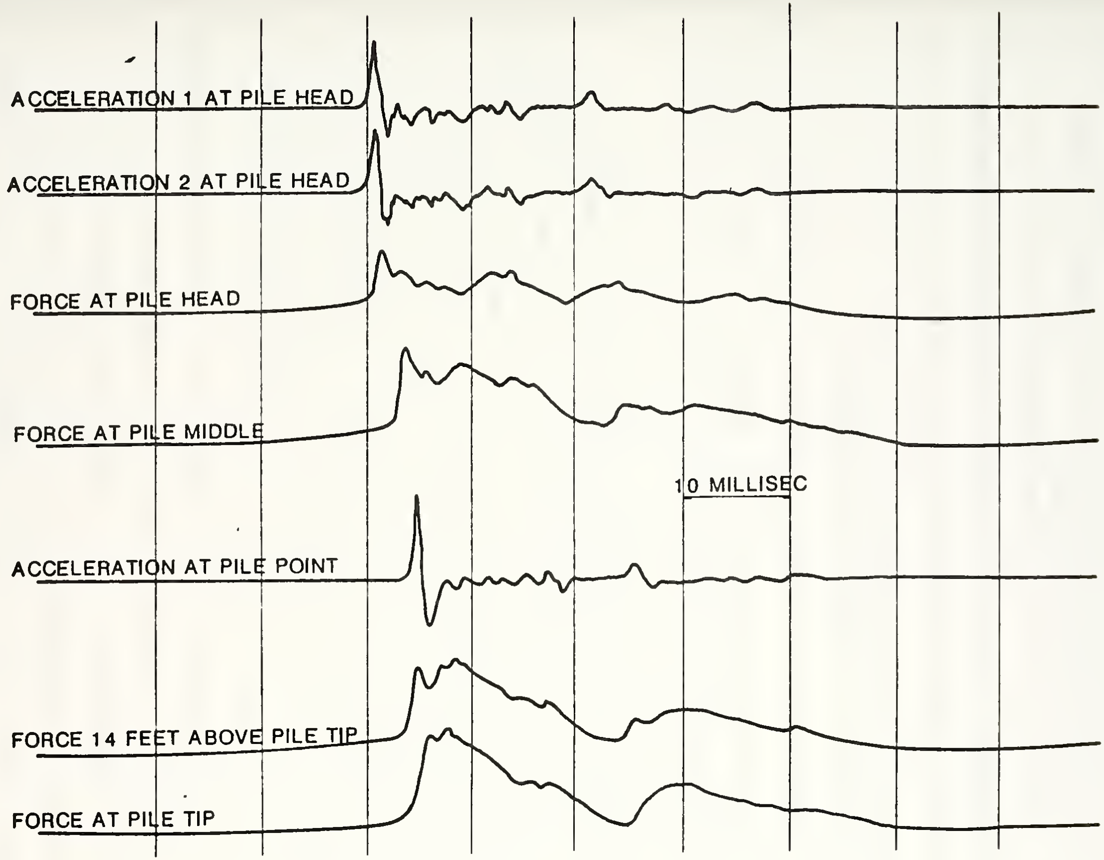


Fig. 34 Sample Result of Dynamic Measurements Taken on a 60 ft. Long Pile (from Goble and Rausche, 1970)

A = cross sectional area

c = speed of the stress wave

If this relationship is not satisfied, then there are inadequacies in the instrumentation system at some point.

3.2.4 Developments in Dynamic Measurements

The instrumentation system outlined above is satisfactory for use by experienced personnel. It hardly represents, however, an acceptable system for routine use by engineers whose primary concerns are with other activities. To solve this problem, effort has been made by Goble and his co-workers to design and develop a special electronic computer and the necessary associated transducers (the Pile Analyzer). Two types of data acquisition and processing can be accomplished using this technique. For the first system, the strain and acceleration signals, after appropriate conditioning, are recorded on a high-speed oscillograph. They can also be recorded on a four channel, portable magnetic tape recorder (Goble et al., 1972), and then taken to the office to be processed and analyzed (using the CAPWAP program as will be explained later). The first system utilizes the special purpose micro-computer. An H-beam force transducer is inserted between the hammer and the pile top. An accelerometer is attached to each side of the transducer. The system is completed by the computer which provides signal condition, analog computation, and a digital readout of pile capacity. This system utilizes approximate equations to allow for monitoring the capacity and pile behavior in situ within reasonable time (e.g. capacity of the pile is displayed directly after each hammer blow). On the other hand, for more accurate analysis, especially if the measurements are to be used for the design of the pile foundation rather than for checking or for quality control, the more sophisticated analysis, using a main frame computer, should be applied. For the field computer system, a sample result is shown in Fig. (35). According to Goble et al (1970), the instrumentation is assembled and all readings taken with only two people.

In addition to the above mentioned measurements, set-rebound measurements are also taken for every blow as usual. In order to monitor the hammer performance, the ram jump height may simply be observed visually, or by using a viedo-tape camera held level with the top of the hammer. This film is to be projected later to obtain the necessary hammer data. Further details on the measurement and processing equipment can be found in Likins (1984) and Rausche (1984).

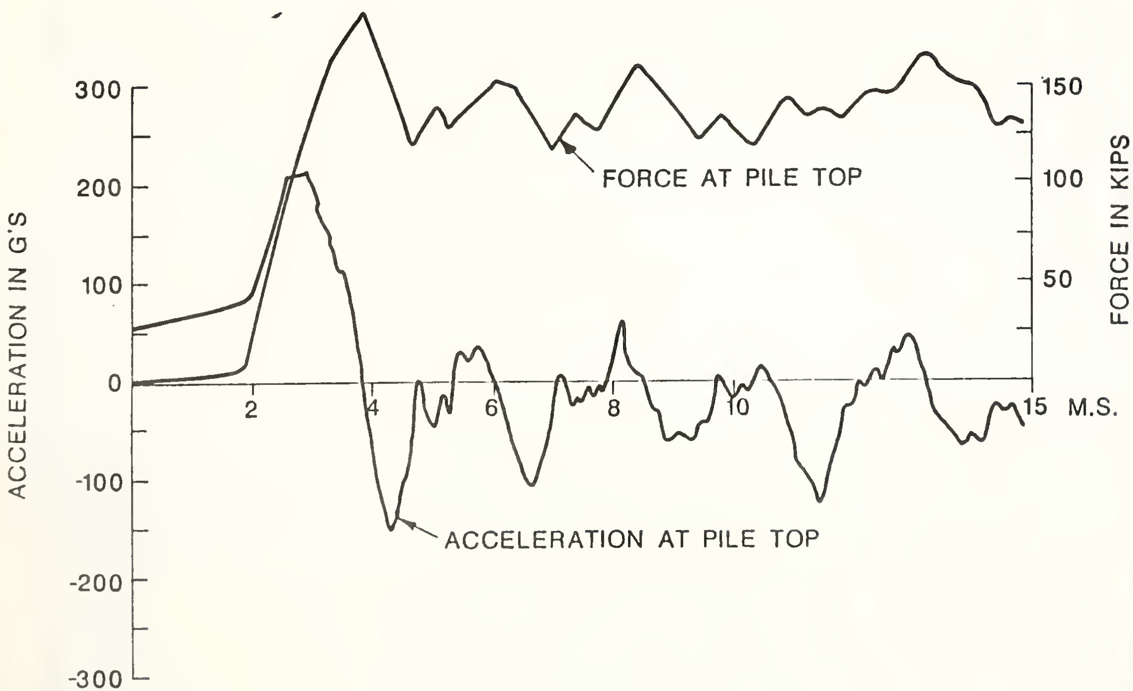


Fig. 35 Sample Output of Pile Analyzer
(from Goble et al., 1970)

3.3 Estimation of the Pile Capacity Using Dynamic Measurements

The idea of using measurements made during pile driving to estimate pile capacity is an old one. Since pile driving causes failure of the soil, it is logical to use measurements made during driving to predict pile capacity. This was the basis of all of the dynamic pile formulas that have been used for more than 100 years. Since only the simplest measurements were possible when the formulas were developed (permanent set per hammer blow), it was natural to use an energy approach. Unsatisfactory results have been obtained because of the very poor representation of all elements of the system, that was necessary in this approach. Attempts have been made to improve these formulas, but there is no indication that improvements have been obtained with the more advanced formulas (Rausche et al., 1985). The results are sometimes not satisfactory due to a lack of knowledge of hammer energy, poor estimates of losses in cushions, inaccurate set measurements, substantial strength change after driving, assumption of a resistance which is constant during the blow and other factors.

With the developments in electronics, it is now practical to measure parameters other than pile set, and these measurements have made improved analysis methods (e.g. using the wave equation) possible. By the 1930's force measurements were already being made at the pile head during driving. Both force and acceleration were measured in the driving system in an extensive pile testing program conducted by the Michigan Highway Department in 1961 (Michigan State Highway Commission, 1965). Beginning in 1964, a research program at Case Western Reserve University collected large volumes of force and acceleration measurements on test piles during driving or, more frequently, during restrike (Goble et al., 1967, 1968, 1970, 1975). This project developed means of analysis to determine wave equation soil parameters acting during impact, using a program known as the CAPWAP (Case Pile Wave Analysis Program), (Rausche and Goble, 1972). As mentioned earlier, the CAPWAP analysis requires a substantial computational effort and, therefore, is usually carried out in the office, with more extensive computer hardware (Rausche et al., 1985).

Several simplified methods of analysis using closed form solutions of the one-dimensional wave propagation theory were also developed using empirical correlations to static pile test results. These solutions can be obtained in real time (using a field computer) for each hammer blow. These methods were improved over a period of time and they have become known as the "Case Method". A rational analysis method has been developed using this approach (Goble et al., 1975).

3.3.1 Simplified Approach Using Field Computers

The basic equation that relates force and acceleration measurements with the static soil resistance is Newton's Second Law. For a perfectly rigid pile struck by a time-varying hammer force, the pile is examined under the action of this force at the instant of zero velocity. The static resistance of the pile R_o^* is found to be (Goble et al., 1970):

$$R_o^* = F(t_o) - M\,a(t_o) \tag{3.2}$$

where: M = mass of the pile

$a(t_o)$ = acceleration at time when the velocity is zero

$F(t_o)$ = force at the top of the pile at that time

Subsequent studies (Goble et al., 1970) have shown that the reliability of the capacity predictions can be improved by averaging the acceleration and force over some time increment around the zero velocity time, in order to average the fluctuations from elastic waves in the pile. Using elastic wave theory, Rausche et al., (1985) derived the following expression:

$$R(t^*) = \frac{1}{2}[F_m(t^*)+F_m(t^* + \frac{2L}{c})] + \frac{Mc}{2L}[v_m(t^*)-v_m(t^* + \frac{2L}{c})] \tag{3.3}$$

where: $R(t^*)$ = total resistance as a function of time t^*

$F_m(t^*)$ = measured force at time t^* (compression positive)

$v_m(t^*)$ = measured velocity of time t^* (downward positive)

M = mass of the pile

L = pile length

c = velocity propagation of the stress wave

E = modulus of elasticity of the material

A = cross-sectional area

It can be seen that the resistance calculation consists of the average of two force values, selected at a time interval 2L/c apart, plus the average acceleration over the same time interval, times the pile mass. This expression reduces to Eq. (3.2) as the interval 2L/c approaches zero (perfectly rigid pile).

One should understand the main assumptions on which Eq. (3.3) was based. It assumes a uniform pile cross section, linear elastic pile behavior, only axial stresses in the pile, and a rigid-plastic soil resistance. The latter assumptions is, of course, not satisfied, which affects the accuracy of the prediction.

According to Rausche et al. (1985), five different types of errors may arise:

1. Capacity is not fully mobilized at time ($t^* + x/c$) as assumed in the solution. This error is avoided by choosing t^* at the time when the major velocity peak occurs (Rausche et al., 1985).

2. Impact energy is insufficient to activate all soil resistance forces. This occurs when the hammer is small relative to the pile capacity. Fortunately, this error results in conservative predictions.

3. The stress wave is short relative to the pile length over which resistance forces act; and resistance forces are, therefore, not maintained at full value during the time period considered. This is a more complicated, but much less frequent error. It occurs when an extremely high shaft resistance causes the pile head to rebound before time 2L/c.

4. Some resistance is velocity dependent (dynamic) and must be subtracted to determine the static capacity. This is a serious error and should be corrected for. Rausche et al. (1985) suggest an approach to deal with this problem, that is, to divide the total resistance to penetration into two parts: a static part and a dynamic part. The analysis using this assumption is given by Rausche et al., (1985).

5. The capacity can change due to setup or relaxation effects. This is a very important consideration in the correlation of dynamic and static load testing. The soil disturbance caused due to driving, together with the developed excess pore water pressure and similar factors, disappear gradually with time. The dynamic testing methods presented in

this report give the static capacity <u>at the time of testing</u>. A practical solution is to restrike the pile after a waiting period, comparable to that given before starting the load test. To obtain the long term service load, restrike of the pile with the longest practical wait is always desirable. If dynamic methods are used at the end of driving and during several restrikes with varying wait times, the capacity can be thoroughly investigated as a function of time.

The analysis presented by Rausche et al., (1985) separates the static from the dynamic resistance results in the following equation:

$$R_s(t_m) = \frac{1}{2}(1-j_c)[F(t_m) + \frac{MC}{L} v_t (t_m)] + \frac{1}{2}(1+j_c)[F(t_m + \frac{2L}{c}) - \frac{Mc}{L} v_t (t_m + \frac{2L}{c})] \qquad (3.4)$$

where $R_s(t_m)$ is the standard result of the "Case" analysis. All the other terms in the equation were defined before, except j_c which is a damping constant. All terms on the right side in Eq. (3.4), except j_c, are evaluated from dynamic measurements. The constant j_c was obtained by correlation of the dynamic measurements with static load test results. Table (11) gives the recommended values of j_c after Rausche et al., (1985). It should be noted that the rather high values recommended for clay are to provide additional conservatism in a soil where less

Table 11. Suggested values of j_c (from Rausche et al., 1985)

Soil type in bearing strata	Suggested range	Correlation value
sand	0.05-0.20	0.05
silty sand or sandy silt	0.15-0.30	0.15
silt	0.20-0.45	0.30
silty clay and clayey silt	0.40-0.70	0.55
clay	0.60-0.10	1.10

experience was available. In general, for any type of soil, higher values of j_c can be used if a conservative approach is required by the engineer. Comparisons with other load test data by Rausche et al., (1985) showed very good agreements between the proposed correlations and the actual test results.

3.3.2 Office Analysis Using CAPWAP

CAPWAP analysis is a more sophisticated one, which also utilizes the solution to the one-dimensional wave equation in an elastic rod to determine the static bearing capacity of a pile.

In the usual dynamics problem where the external boundary conditions are known, either force or acceleration is used as input, and the other quantity is then calculated by a dynamic analysis as the output. In the CAPWAP analysis, both force and acceleration records are used as input information, and the characteristics of the soil resistance are calculated as the output, including static and dynamic values. This is done by using one of the two records (force or acceleration) as input, and soil resistance parameters are adjusted until the computed output matches the other measured quantity. Since it is intended to predict forces exerted by the soil along the pile, it is natural to compute the force on top of the unsupported pile of length "L" using the velocity record as an input. The difference between measured and computed force is the force due to the soil action.

The CAPWAP analysis can also be used to predict the load transfer along the pile, i.e. the distribution of shaft loads along the pile. It can also be used to predict a load versus deflection curve. Comparisons between predicted and measured results from static load tests did not show a very good agreement. This is primarily due to the inadequacy of the soil model used in the analysis. This is clearly shown in Fig. (36). Table (12) gives some data which show the difference between the prediction and the actual capacities. According to Goble et al., (1970), the correlation between predicted and measured capacity is better for the piles driven in coarse grained soils than those in clayey soils.

In order to record and store the data to be used for CAPWAP, a magnetic tape recorder is required. It stores an electronic image of the event measured, and should have the capability of recreating it at a later time. This makes a fully automatic data processing system possible. Later in the laboratory, the analog tape recorder can be converted to digital form, some simple computations can be made on the digital data during the conversion operation, the data can be stored on digital magnetic tape for later conversion, and a plotted record can be obtained on a computer controlled plotter.

After acquiring experience in dynamic measurements, the engineer would be able to reach some interesting conclusions just by looking at the measurement output. For example, Fig.

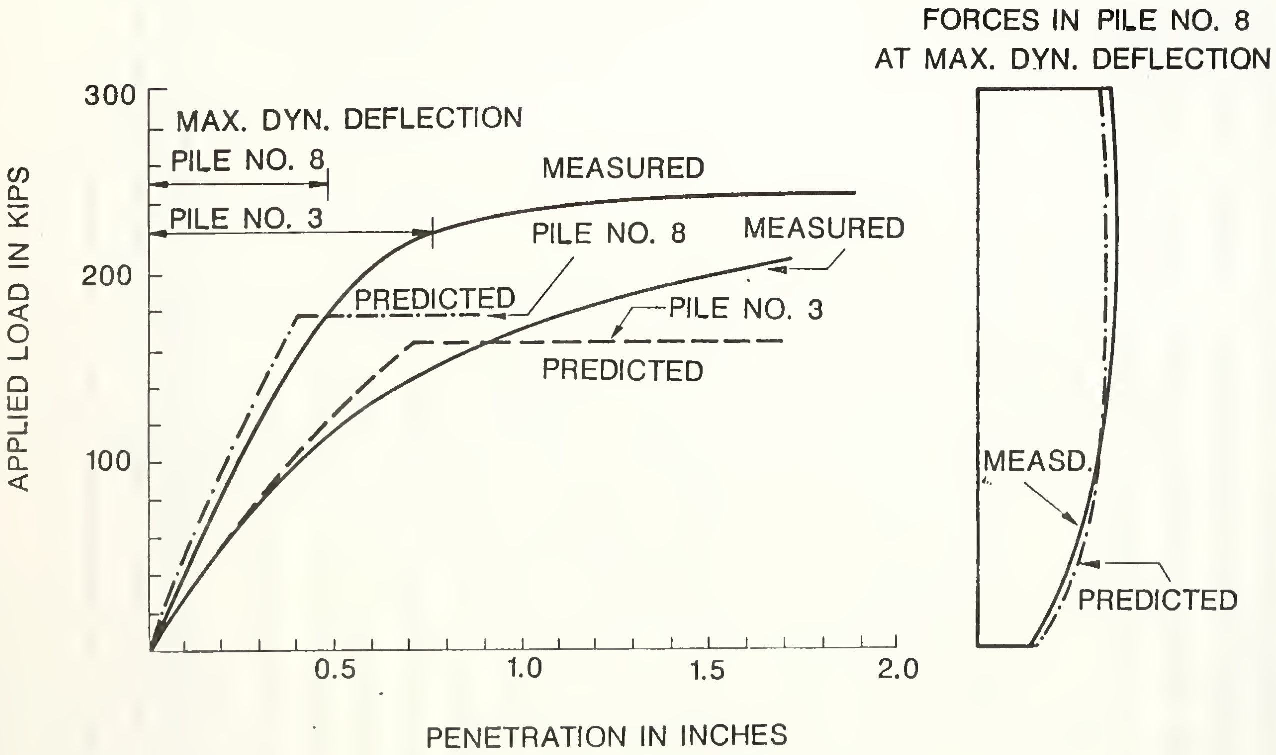


Fig. 36 Comparison of Predicted with Measured Load versus Penetration Curve and Predicted and Measured Forces in Pile at Maximum Dynamic Deflection (from Goble et al., 1970)

Table 12 Comparative Results from Automated Prediction Method (from Goble et al., 1970)

1 Pile No.	2 Static Capacity from Load Test At max d* (kips)	3 At Ultimate (kips)	4 Total Static Capacity Predicted (kips)	5 Max. Dyn. Resistance Predicted (kips)	6 Static Res. Predicted For Pile Tip (%)	7 Static Res. Measured For Pile Tip (%)
3	145	198	160	15	40	-
4	90	104	91	45	17	33
5	180	224	163	43	70	42
6	170	228	234	48	44	45
7	180	204	210	29	85	50
8	170	242	185	52	75	45
9	170	226	167	20	80	-
10	165	210	163	56	58	-
11	200	222	231	16	65	-
12	66	70	76	55	0	46
13	94	96	119	75	32	21
14	38	44	56	67	5	55
15	80	86	121	107	38	43
16	92	94	151	121	19	-
17	136	176	193	71	37	-

* Value from load versus penetration curve equal to maximum displacement under hammer blow. All piles were of 12-inch diameter hollow steel pipes.

(37) shows the force and velocity traces obtained by dynamic measurements for two conditions. The first is obtained at the end of initial driving and the second is obtained at the end of restriking the pile two weeks later (Fellenius, 1984). Since the difference between the force and velocity curves gives an indication of the static soil resistance (refer to Eq. (3.2)), the second graph indicates a tremendous gain in soil capacity due to the set-up factors. This observation was confirmed by static pile load tests. On the other hand, Fig. (38) shows no set-up effects for another case, since the difference between the velocity and force traces remains almost unchanged after restriking the pile. Another interesting case is shown in Fig. (39). In some soils, relaxation can occur, i.e. a decrease of the pile capacity after some time after driving. Fig. (39) indicates an end bearing pile with little or no shaft resistance. This is indicated by the second and largest force peaks which are the effect of the force wave reflecting from the pile toe on the bedrock. In comparing the two force traces, i.e. after initial striking and after restriking, it is evident that the wave reflected at restriking is weaker than the one at the end of the initial driving. This observation, of course, will allow the imposition of a solution to this problem to get the pile into the required capacity.

In summary, dynamic measurements have been used to predict the geotechnical pile capacity, either in-situ using a field com-

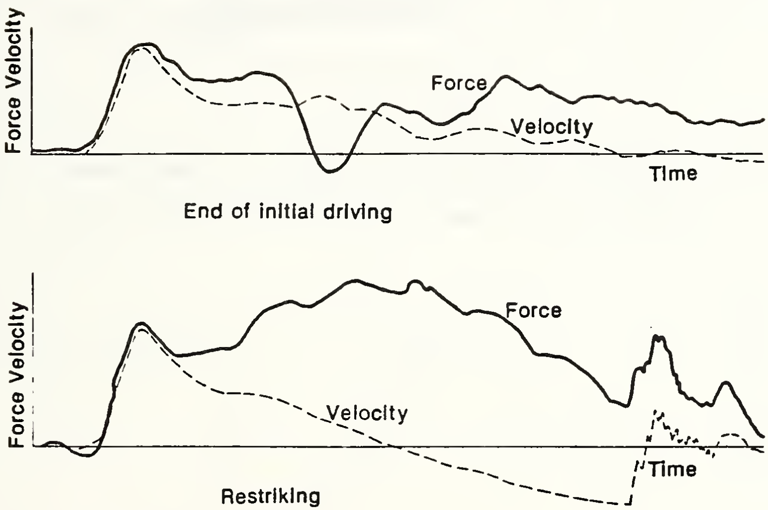


Fig. 37 Effect of Soil Set-up on Wave Traces
Measured at End of Initial Driving
and at Restriking (from Fellenius, 1984)

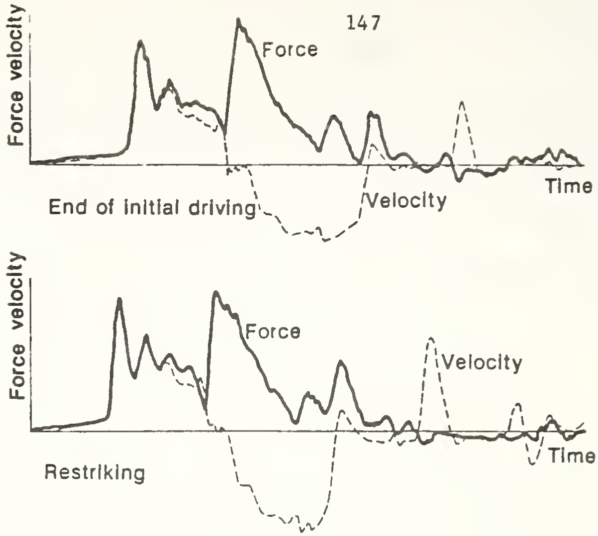


Fig. 38 Effect When No Soil Set-up is Present on Wave Traces Measured at End of Initial Driving and at Restriking (from Fellenius, 1984)

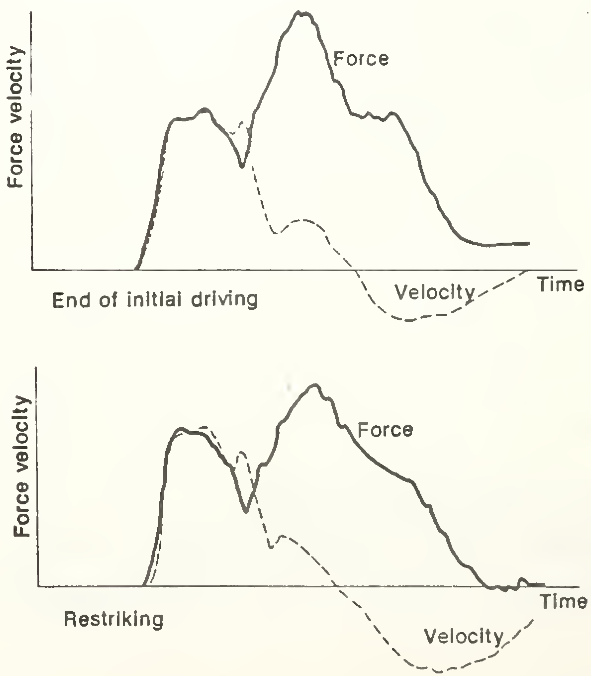


Fig. 39 Effect of Relaxation of Toe Resistance on Wave Traces Measured at End of Initial Driving and at Restriking (from Fellenius, 1984)

puter, or in the office using a more sophisticated analysis (CAPWAP). The latter analysis can also be used to predict the load transfer along the pile shaft and the load-deformation curve that would be obtained from a pile load test. Experience, however, indicated only a moderate success in these two predictions. This is primarily due to the lack of a good soil model. Also the predicted capacity should not be directly used in the design without verification using other techniques like dynamic formulas, reliable static formulas or pile load tests. The measurements have been very successful in calculating the wave equation parameters and detecting certain phenomena after driving, e.g. soil set-up or soil relaxation. Therefore, using the measurements with the wave equation analysis is much better than just using the wave equation with the arbitrary assumption of parameters which may not be close enough to reality.

3.4 Evaluation of the Performance of Pile Driving Systems Using Dynamic Measurements

The understanding of pile driving and pile driving hammers is difficult. First because of the interaction of many components such as ram, impact block, cushion assembly, pile and soil, and second because of the very short duration of the total impact phenomenon. Also, a wide variety of hammers is commercially available and the performance and characteristics of each of them are quite different. Experience and engineering judgment have been the best tools to evaluate this problem. However, problems such as pile overdriving or damage frequently occur and unnecessary blow count requirements lead to losses of time and money.

To better understand this situation, force and acceleration measurements have been used, by means of simple theoretical models, for a meaningful evaluation of quantities such as energy transfer and forces in hammer components.

Dynamic measurements, although originally meant for pile capacity prediction, do provide useful data for the computation of energy delivered by the driving system to the pile. Of particular interest is the aspect of hammer performance for an open-end diesel hammer, since widely differing views are held of its operating characteristics (Goble et al., 1972).

In order to do that, the ram stroke, force, acceleration and displacement are recorded simultaneously as described before.

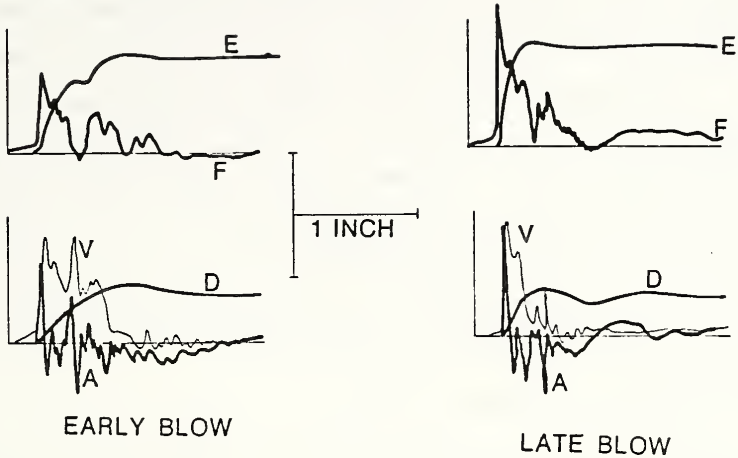
The video tape records are processed manually to obtain the ram stroke for each blow. The automatic data processing system is used to calculate the delivered energy. The processing system operates as follows: the measured analog strain record is converted to digital form and then to force. The two acceleration records (from the two accelerometers) are digitized at the same time and averaged to obtain a single signal. The digital acceleration and force records for each blow are stored on digital magnetic tape for later processing. Calculation of energy is accomplished by integrating the averaged acceleration, once to obtain velocity, and a second time to obtain displacement. The final displacement is automatically checked with the measured set, and if a difference is observed, the acceleration zero is adjusted to correct the final displacement. Also, the first velocity peak is checked for proportionality with the first force peak. If this proportionality is seriously violated, the results from that specific blow are ignored. It should be noted that the peak accelerations measured at impact are usually extremely high, in some cases over 500 g´s, where g is the acceleration of gravity (Goble et al. 1972).

Energy is calculated from the following expression:

$$E(t_1) = \int_0^{t_1} F(t)\, v(t)\, dt \tag{3.5}$$

where E is the energy at the pile top expressed as a function of time, and other quantities have been previously defined. Zero time is taken as some time before the beginning of the blow. Thus, an energy-time record can be obtained which can be automatically plotted. Typical plots of such relations are shown in Fig. (40), together with force, velocity and displacement curves. To demonstrate differences caused by resistance changes, plots of both an early and a late blow are shown. It is important to note that in the early blow the maximum value of energy transfer is reached at a later time after impact and no energy is transferred back to the hammer (the curve stays horizontal). In the later blow a peak value is reached after which the energy curve decreases, indicating that some energy is returned to the hammer. Comparison with the manufacturers rated value evaluates the hammer performance.

It should be noted that the energy considered is the value of the energy that was actually transferred from the ram to the pile during the blow. Obviously, the total energy transferred is the final value, i.e. a value at a time when the energy curve stays constant. For constant fuel injection with the same cumbustion efficiency and with the same hammer losses, this value should be constant, or, in case not all of these restrictions are valid, it should be at least independent of the ram stroke (Goble et al., 1972). Results shown in that reference indicate that there is only a 3 percent chance that the ram stroke and energy are related (for the open-end diesel hammer). Rausche and Goble



Scales

Horizontal:
Time 1 in. = 25 msec

Vertical:

A- acceleration	1 in. = 400g
V- velocity	1 in. = 10 ft/sec
D- displacement	1 in. = 2 in.
E- energy	1 in. = 10 k-1
F- force	1 in. = 400 k

Fig. 40 Sample Plots of Automatically Processed Data (from Goble et al., 1972)

(1972), based on actual measurements, showed that the driving ability of a hammer depends on the impact velocity at the pile top and the pile cross sectional area. The velocity at the pile depends on the cushion, energy losses in the hammer, the ratio of ram mass to anvil plus cap mass and in the case of diesel hammers, on the hammer combustion chamber pressure before impact. The engineer, however, should be cautious when dealing with the driving process since large portions of the hammer energy can be stored in the pile and the driving system without producing pile penetrations. Applications of energy formula, therefore, have to be considered skeptically even if the exact hammer energy is known (Rausche and Goble, 1972).

Before dynamic measurements, the criterion for controlling tension stresses in piles during driving was based on the pile-ram weight ratios. Measurements showed that this criterion is quite unsatisfactory when applied to diesel hammers (Goble et al., 1976). In that study, the highest measured tension stresses were generated by the heaviest ram (the opposite could have been predicted based on the pile-ram weight ratio). Therefore, if current pile-ram weight ratios are used for selecting driving equipment, damage problems will be more frequent and severe. Dynamic measurements also showed that the diesel hammers are less likely to damage concrete piles by the induced tension forces due to their low velocity in easy driving.

Dynamic measurements can also be used to evaluate the efficiencies of different operating pressures, strokes or batters, or changes in hammer maintenance conditions by comparative testing of hammers of one type or of a single hammer over an extended period of use. Relative efficiencies of different hammer types can also be evaluated. They are also useful in examining any preignition problems with diesel hammers. Also if some unexplained blow count is recorded, possible changes in hammer performance can be detected. All of the above uses are possible via the hammer energy output measurement described before.

Furthermore, the driving system as a whole can be checked by the dynamic measurements, i.e. the cushions, cap blocks, ..., etc. It is well known that one of the major advantages of the wave equation analysis is the representation of the elements of the driving system. The characteristics of these elements, however, may change as driving proceeds. The dynamic measurements are very efficient in checking the effects of changes in cushion properties or helmet assembly by their effect on transmitted energy and forces in the pile.

3.5 Evaluation of the Pile Performance Using Dynamic Measurements

Dynamic measurements have proven to be very useful in evaluating the pile performance after driving. In this aspect, three main characteristics can be checked using the measurements, namely the driving stresses at the measuring location, the total length for existing piles where the lengths are unknown and the extent and location of pile structural damage.

Since the force is measured directly at the pile top, stresses can be obtained and checked very easily, i.e. excessive compressive stresses that may exceed the material capacity or excessive tensile stresses that may cause severe tension cracking. Using the wave equation analysis with the measurements permits the evaluation of stresses along the whole pile shaft. Various hammers, cushions, cap blocks, ..., etc. can be used and checked with the measurements to minimize the undesired excessive stresses. Fig. (41) shows a typical trace of compressive and tensile stresses in the pile, related to the cumulative blow number, which can be obtained from dynamic measurement.

Sometimes the actual pile length is not known for some reason, and it may not be known whether the pile is supported at the required depth or not. Dynamic measurements can be used to determine the pile length (L). The process is illustrated in Fig. (42). Suppose that the velocity record of a friction pile is as shown. If the time between two successive peaks is

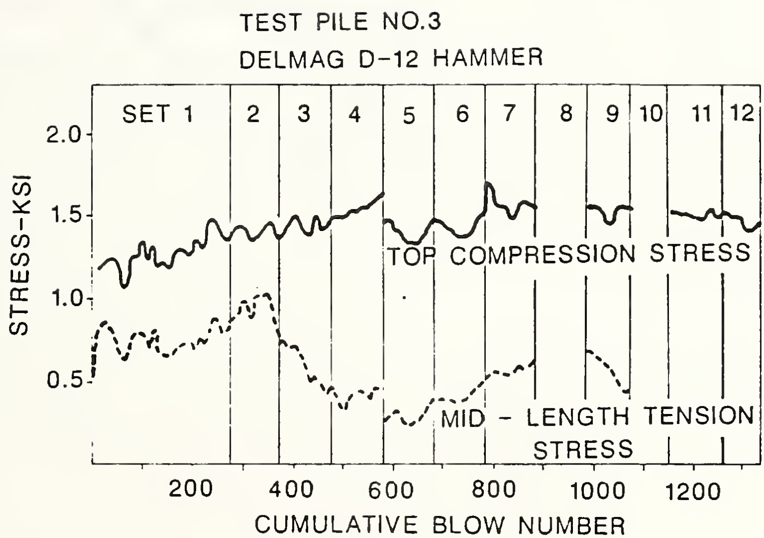


Fig. 41 Maximum Stresses for a Test Driven Pile (from Goble et al., 1976)

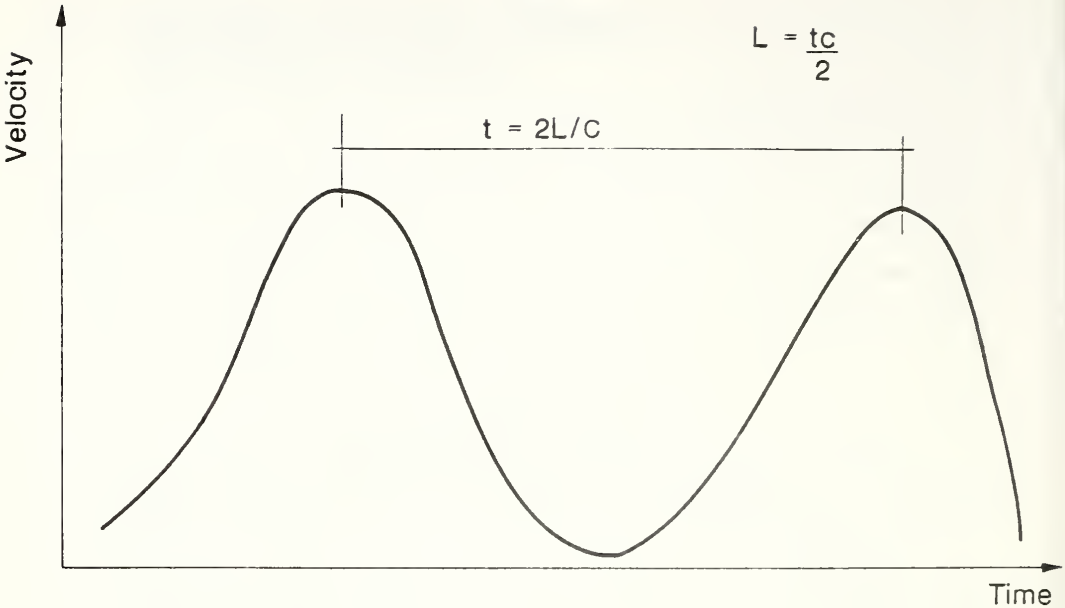


Fig. 42 Determination of Pile Length from a Velocity Graph for a Friction Pile

recorded (t), the length of the pile can be obtained from the relationship:

$$L = \frac{c\ t}{2} \tag{3.6}$$

where c is the wave velocity.

The same principle can be used to detect any cracks in the pile. If the pile was completely intact, a velocity peak would be observed after two units of L/c from the previous one. This is a result of wave passing and reflection over the entire length of the pile. However, if a crack exists at some distance (Δ) from the pile top, the wave would be reflected earlier at the position of the crack, and a "blip" would be observed after some time (t_x). This is illustrated in Fig. (43) which shows the velocity record for an end bearing pile, as the velocity is plotted against L/c (time units). If (t_x) is determined from this graph, the distance (Δ) can be obtained using the relationship:

$$\Delta = \frac{t_x\ c}{2} \tag{3.7}$$

Hence, dynamic measurements could be used as a quality control procedure for driven piles as the pile is being driven or re-struck. This is again another successful aspect of these measurements.

After the brief summarization of the potential uses of dynamic measurements, the reader may refer to studies in which

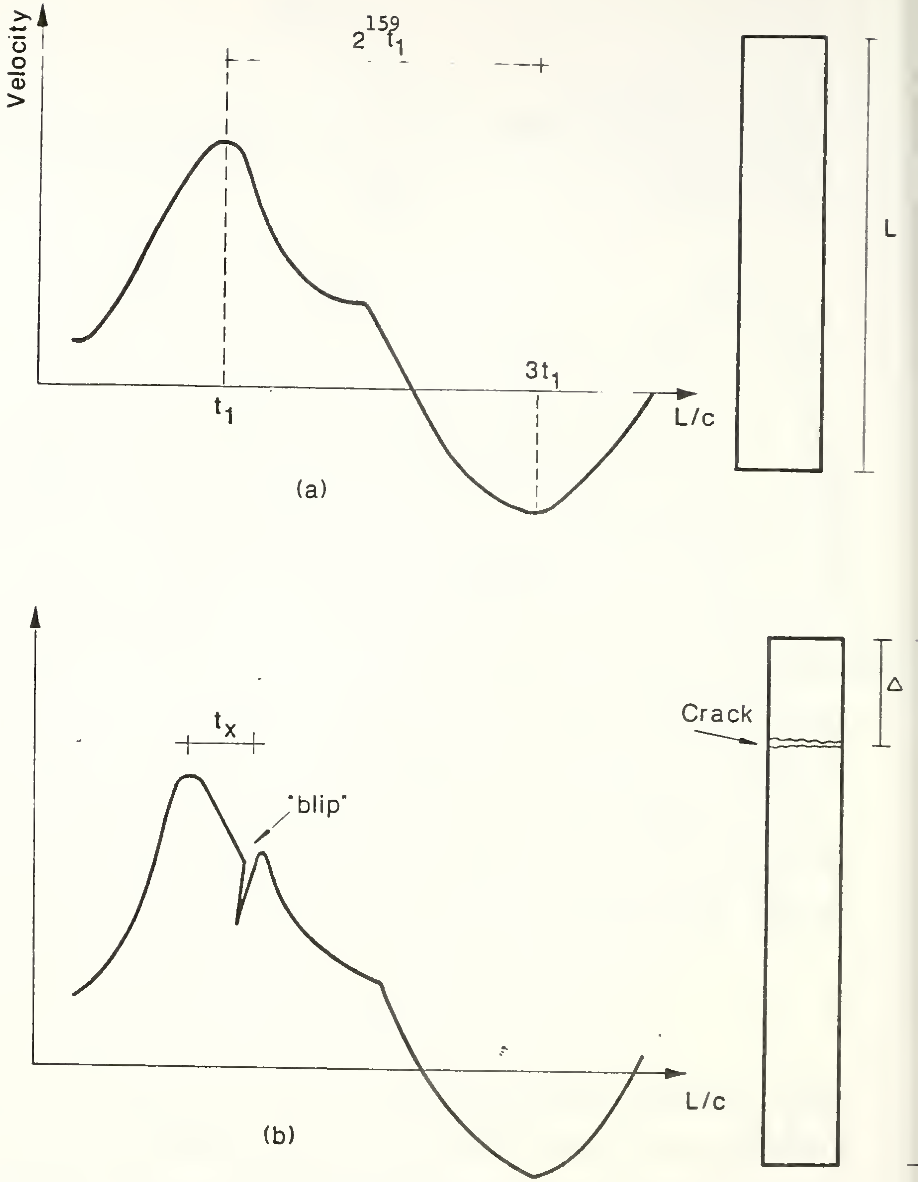


Fig. 43 Quality Control of an End-Bearing Pile Using Dynamic Measurements:

(a) intact pile

(b) pile with a crack

these measurements were applied successfully (e.g. Goble et al., 1972; Fellenius, 1984; Baker et al., 1984; ..., etc.). One of the projects in which the dynamic measurements were applied successfully is the 1980 Indiana Toll Road improvements project. Applications that have been discussed in this report were used for many piles. For example, the piles PB-3j, PB-2B, PB-2I (ATEC, 1982), and the piles C-2, B-7 (ATEC, 1983) were tested during initial driving and during restriking to estimate their capacities and the induced driving stresses, to determine that the piles were not damaged. The performance and efficiency of the hammer used for driving the piles were monitored and evaluated (ATEC, 1983). All these applications have been successful, and have resulted in better design and driving criteria.

3.6 Conclusions and Recommendations

Dynamic measurement techniques are the best ones introduced thus far for monitoring the pile during driving. They can be applied, together with the wave equation analysis, in a variety of aspects. Among their uses are the prediction of pile capacity and load transfer; the evaluation of the driving system with respect to the hammer efficiency and performance, cushions, cap blocks, etc.; the measurement of the pile stresses; and the quality control of the piles. Prediction of pile capacity by this technique is of course much better than the driving formulas (e.g. ENR, Hiley, etc.). On the other hand it is not accurate enough to be used alone without other techniques of pile design. This is primarily due to the insufficiency of the soil model used. The prediction by this technique, however, completes a spectrum of approaches for pile design, and gives more confidence in the other procedures, since it is based on direct measurements during the driving process. On the other hand, dynamic measurements have proven to be quite successful in determining the wave equation parameters, measuring the actual energy delivered to the pile, monitoring the driving process with all its elements, measuring the pile stresses and performing quality control procedures.

One of the purposes of this report was to introduce the subject of dynamic measurements to the IDOH in an easy-to-follow manner. Some of the complicated mathematical derivations and expressions were omitted to help simplify the subject. The main purpose was to show the potential uses of these measurements and to illustrate selected theoretical background points which are associated with the subject.

As was shown in this report, dynamic measurements have proven to be very useful in many aspects. Hence, it is recommended that the IDOH use them in the operations involving pile driving. This can be achieved through two steps:

1. In the beginning, they can be used with the help of consulting expertise for selected important jobs. In the meantime, personnel can be trained to be familiar with the technique and eventually use it with no consulting help.

2. After the personnel have been well trained, the IDOH can purchase the equipment, the price of which is decreasing, and use it for most of its jobs, or even offer consulting on its use for any interested potential user. The savings that can be achieved from the various jobs would very soon cover the price of the equipment. More importantly will be the improvement in design and execution procedures of pile foundations, which will not only produce savings on the short term, but will also reduce maintenance and replacement costs in the long run.

A list of the required equipment, with all the options, along with the prices given by Pile Dynamics, Inc. are given in Table (13). For further information, the reader may refer to Pile Dynamics, Inc., 4535 Emery Industrial Parkwa, Cleveland, Ohio 44128. It should be noted that the average dynamic test takes about 20 min. including the preparation of the pile for testing and attaching the necessary instrumentation. In this case, if all the equipment and the personnel are available, the test cost/pile would be in the order of $200.00 (according to Pile Dynamics, Inc.). This does not include performing the CAPWAP office analysis.

Regarding the personnel required for this procedure, two people can take care of performing the test. One of them is an engineer who takes care of the data acquisition system and the readout of the pile capacity. The other one is a technician who is responsible for fixing the instruments to the pile and other relevant technical details. According to Pile Dynamics, Inc., the use of the latest simplified equipment (refer to item 3.2.4) does not require a lot of training. The average engineer or technician could be easily trained to perform the job satisfactorily.

It should be noted that the described dynamic testing is not a substitute for the classical static load test, which gives a most realistic indication to the static pile capacity. As stated earlier, the pile capacity estimated from dynamic measurements

Table 13. Measurement System Price List
(effective March 1, 1986)

Item	Description	Unit Price
1	Pile Driving Analyzer, Model GB, including two main cables at 70´ each, six BNC interconnectors, two breakout cables (from transducers to main cable), operating manual and transit case - and one-year warranty	$36,000.00
2	Optional RS 232 interface board for PDA data transfer to computers or plotter	$1,500.00
3	Optional PDA software (hardwired) for field plotting on HP 7470A digital plotter	$500.00
4	Accelerometer (5000 g limit) for steel piles recommended quanity - 6	$450.00
5	Accelerometer (2000 g limit) for concrete piles recommended quantity - 6	$400.00
6	Strain Transducer 350 ohm wheatstone bridge bolt-on type, recommended quanity - 6	$300.00
7	Connection Adapter for back-up	$100.00
8	Oscilloscope, dual-trace with x-y e.g. Hameg 203-4	$750.00
9	Tape recorder, 7 channel, FM plus voice TEAC R71 cassette recorder.	$6,000.00
10	Transit case, recommended quanity - 2	$300.00
11	Five day training session by experienced civil engineer - to be quoted Usual charge within U.S. for one consecutive week	$3,000.00

Prices subject to change without notice.

PILE DYNAMICS, INC.
4535 EMERY INDUSTRIAL PARKWAY
CLEVELAND, OHIO 44128

still has the limitation of the inadequacy of the soil model used in the analysis. More research is still needed to introduce more realistic soil models into the wave equation analysis to get better predictions of the pile capacity. Nevertheless, dynamic measurements have the advantage of being cheaper and faster and they also allow the engineer to check a larger number of piles in one site. The best technique would be to perform a static load test and correlate its results with the dynamic predictions, and then use this correlation to examine a larger number of piles using the dynamic measurements.

CHAPTER 4

RESIDUAL STRESSES DUE TO PILE DRIVING

4.1 General

A very important phenomenon related to pile-soil interaction is the existence of residual stresses for driven piles after driving. The discussion of this phenomenon in the literature has started relatively recently. The first paper that described residual stresses effects and presented approximate techniques for their evaluation was published by Hunter and Davisson (1969) using data from pile tests performed at Lock and Dam No. 4, Arkansas River Project. Since then, detailed studies of load transfer from load tests have indicated the presence and effect of residual loads after driving. The first study that used direct measurements for residual stresses was published by Gregersen et al., (1973).

The generation of residual loads is a function of the load-unload mechanisms of pile installation. During the pile's downward movement, the pile-soil friction is acting upward on the pile to resist the pile penetration. The soil point resistance is also acting upward. During the rebound that follows, the soil under the point pushes the pile upwards while the pile tends to return to its initial length elastically. The two components of the rebound create enough upward movement to reverse the direction of the pile-soil friction, which then acts downward, at least in the upper portion of the pile. This negative friction is counterbalanced by residual skin friction in the lower portion of the shaft and also, if the applied compressive load was high enough, by residual point load on the pile. Equilibrium is reached when enough of the friction stresses have reversed themselves in order to keep the bottom of the pile stressed against the soil (Vesic, 1977; Briaud et al., 1983).

Because the impact driving process consists of periodical loading and unloading of pile head by dynamic impulses, driven piles usually contain substantial residual loads, the existence of which has an effect on load-settlement response of the pile. More careful interpretation of pile load test results may be needed to avoid undue conservatism in pile foundation design, especially for piles in cohesionless soils. Since the ordinary limit equilibrium analyses of bearing capacity do not account for the existence of residual stresses, using the resulting static formulas alone, neglecting these stresses, may have serious design implications. Another effect of neglecting residual stresses is the incorrect prediction of pile drivability and blow counts. This may result in important installation difficulties which should be otherwise anticipated (Briaud and Tucker, 1984-a).

A qualitative representation of the difference between "true" loads (including the effect of residual stresses) and measured loads in conventional load tests is given in Fig. (44).

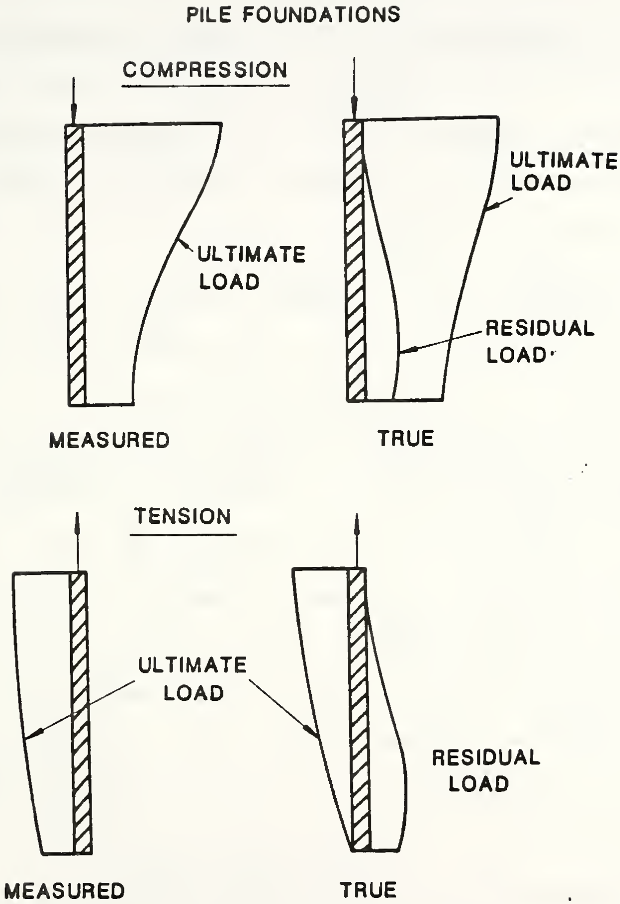


Fig. 44 Difference between True Loads and Measured Loads in Conventional Load Tests (from Briaud and Tucker, 1984)

Also, Fig. (44) indicates that the presence of residual loads causes an apparent concentration of skin friction in the upper part of the shaft, if the pile is loaded in compression. Conversely, for piles loaded in tension, after being loaded in compression or immediately after driving, the apparent load distribution shows a tensile load at the pile tip equal in magnitude to the residual point load.

A simple illustration of the effect of residual stresses on the distribution of unit skin friction and load transfer along the pile shaft is given in Fig. (45) (Holloway et al., 1975). If, for simplicity, the distribution of unit skin friction along the pile shaft is assumed to be linear, as predicted by static formulas, without any consideration of arching or crushing of particles below tip, the resulting curves are denoted by 1 and 2 for compression and tension, respectively. The corresponding load transfer (Q vs. depth) can be obtained from the fact that its slope is proportional to unit skin friction distribution, as explained earlier in the discussions. The actual load transfer, as observed in most load tests performed for piles driven into cohesionless soils, is indicated by curves 1M and 2M for compression and tension tests, respectively (provided that instrumentation was zeroed after installation and prior to the load test). The differences between the two sets of curves (1, 2 and 1M, 2M) give the assumed distribution of residual loads along the pile shaft (curves 0 and 0′). The above discussion illus-

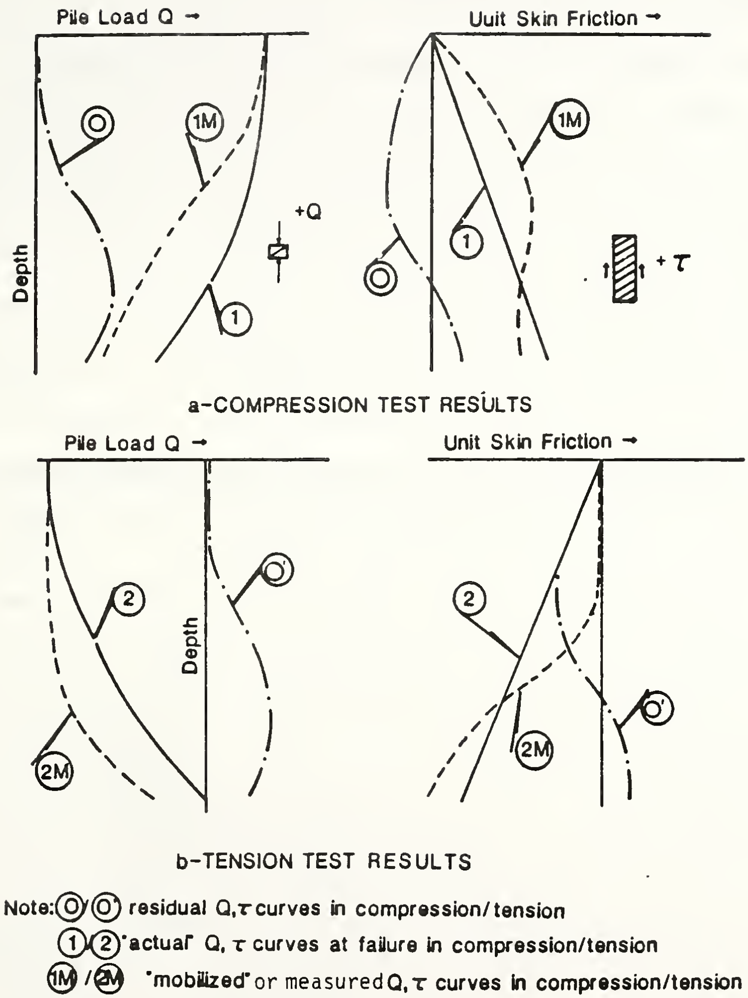


Fig. 45. Residual Load Distribution (from Holloway et al., 1975)

trates one explanation for the observed distribution of unit skin friction in pile load tests.

It should be noted that the total loads transferred by the pile are the same whether residual stresses are considered or not, since the pile is in equilibrium after driving and before loading (i.e. the summation of shaft and tip residual loads should be zero). However, the load test data can not be extrapolated to other pile lengths if the actual load transfer is not known. Also there are many other reasons for the accurate determination of load transfer (Leonards, 1985). Therefore, it is of vital importance to determine the residual stresses, which is the only way to get a correct load transfer for driven piles.

Another very important effect of the residual stresses is that they may result in a substantial reduction in pile settlement. This is due to the concentration of skin resistance in the upper portion of the shaft (Vesic, 1978). Approximate analysis performed by Vesic, which incorporated the residual stress effect for two major projects which involved driven piles for the foundation, led to settlements prediction in the order of 1/6 - 1/4 of the predicted settlements neglecting the residual stresses. Load tests fully confirmed this assumption. It should be noted, however, that although the single pile settlement decreases due to considering the residual stresses, the group settlement ratio (which is the ratio between the settlement of the pile group and a single pile) increases as the load carried by friction increases (Leonards, 1972). This might lead to an increase of the pile group settlement. These examples indicate clearly the importance of introducing the residual loads in the prediction of single pile and pile group settlements. They also demonstrate the doubtful value of numerous theories of pile settlement behavior published in the literature in recent years, which do not consider this phenomenon at all (Vesic, 1978). It seems necessary to either modify these theories to incorporate the residual stress effect, or to develop new techniques on a sound theoretical basis, rather than applying simplified or empirical approaches.

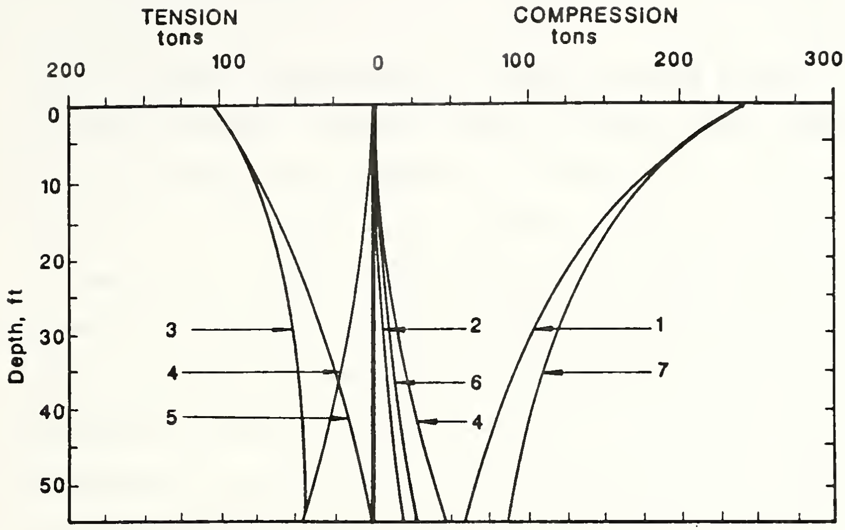
4.2 Current Methods Used for Residual Stress Measurement

Four major techniques have been used to obtain the residual pile loads from load test data:

1. The direct way to get residual stresses is to instrument the pile and to zero the instrumentation before driving (Gregersen et al., 1973). In this case, the instrumentation would be of the strain gage type. Load cells could also be placed under the pile tip for direct measurements of the residual point pressure (e.g. see Gurtowski and Wu, 1984). However, there is a major doubt as to the effect of driving on the sensitivity of the strain gages. The readings after driving may be of no meaning due to this effect. Reliable instruments need to be developed to correctly

measure the initial pile stresses prior to load testing, without being affected by the installation procedure. This technique has been used successfully for tests on model piles driven into cohesionless soil to obtain the "true" load transfer (Yazdanbod et al., 1984).

2. Hunter and Davisson (1969) used a certain testing sequence in order to determine the residual loads. The pile is driven and the instrumentation is zeroed afterwards. Next, the pile is loaded in compression to failure and then unloaded. After the gages are read, they are zeroed again. The final step is to load the pile in tension up to failure and to unload it. To illustrate the reasoning behind this technique, Fig. (46) is introduced. Curves (1) and (3) represent the measured ultimate compression or tension load distribution assuming no stresses in the pile at the start of the test. Curves (2) and (4) represent the residual load in the pile after full release of compression or tension load due to these loads, plus the original residual loads due to driving. If it is <u>assumed that tension loading results in no residual loads</u>, then curve 4 represents the residual compressive loads due to both driving and compression testing. Therefore, the adjusted (true) tension load distribution is given by curve 5, which is the difference between curves 3 and 4. If curve 2 is subtracted from curve 4, the result would be curve 6, which



1 - Ultimate Compression
2 - Unload Compression
3 - Ultimate Tension
4 - Unload Tension
5 - Corrected Tension
6 - Residual Load due to Driving
7 - Corrected Compression

Fig. 46. The Hunter-Davisson Method to Obtain Residual Stresses (from Briaud and Tucker, 1984)

represents the residual loads due to driving only. If curve 6 is added to curve 1, the result would be curve 7, which is the adjusted (true) load distribution in compression.

3. The third method is used if instrumentation was not read after compression and tension loading (Briaud and Tucker, 1984-a). This means that curves 2 and 4 shown on Fig. (46) are not available. Hence, it is assumed that no residual stresses result from compression loading of the pile. Therefore, the tip load measured in tension is assumed to be the residual driving tip load, and the residual load distribution is assumed to be linear from zero at the pile top to the residual tip load. Obviously the assumption of this method is not correct and leads to further errors in using the Hunter and Davisson method.

4. The least reliable method is based on an assumption that the side friction is the same in tension as in compression. This is done if only the butt and tip displacements are measured without instrumenting the whole shaft. In this case a tension test subsequent to the compression test is necessary. The tip load is thus the difference between compression and tension loading results. In this way, nothing is known about residual stresses although the "true" load transfer is obtained (since it is included in both the tension and compression results assuming they are

the same). Obviously all the assumptions related to this approach are incorrect. In addition to the previously stated assumptions, it was demonstrated earlier that assuming the same friction in tension as in compression is incorrect. Therefore, it is impossible to get at least an approximate distribution of the residual loads without instrumenting the pile.

The demonstration of the available four methods that have been used to measure residual stresses indicate that none of them can be used to get the exact or true load transfer, that includes the residual stress effect. A possible alternative is to get the residual point load, which can be obtained fairly accurately as explained earlier using a load cell at the tip or using a pulling test after the compression test, then to try to get the residual friction loads that balance the residual tip load, and assume a reasonable distribution for these loads.

4.3 Review of Research on Residual Stresses for Piles in Cohesionless Soils

Little research has been done on residual stresses and their effect on the pile-soil interaction quantitatively. Hunter and Davisson (1969) used their suggested procedure for evaluating the residual loads to obtain the adjusted (true) tip and friction ultimate loads for compression and tension load tests. Table (14) gives a summarization of the adjusted values and the percen-

Table (14) Adjustment of Load Test Results by Considering Residual Loads (from Hunter and Davisson, 1969)

Test Pile No.	Type of Pile	Total Load (tons)	Tip Loads Original (tons)	Adjusted (tons)	Change (%)	Original (tons)	Adjusted (tons)	Change (%)	Tension Loads (tons)
1	12.75 in. OD	172	48	85	+77	124	87	-30	92
2	16.0 in. OD	251	75	120	+60	176	131	-26	116
3	20.0 in. OD	258	112	160	+43	146	98	-33	120
7	14 BP 73	220	65	90	+23	155	130	-16	75
10*	16 in. OD	228	83	80	-4	135	138	+2	110
16**	16 in. OD	165	50	90	+80	115	75	-35	73

* Driven with Bodine resonant driver.

** Jetted to 40 ft.

tage change in tip and friction loads after considering the residual loads. A large scatter is easily observed for these percentages, especially for the change in tip load. It is interesting to notice that using a vibratory hammer resulted in almost no residual loads (as also observed by Mansur and Hunter, 1970; Holloway et al., 1975). This is to be expected, since resonant pile driving temporarily displaces the soil, and penetration of the pile is achieved under the static weight of the pile and hammer. The vibratory driver causes penetration by transmission of high frequency, low amplitude longitudinal stress waves in "resonance" with the pile-soil system. Analysis of more test data made by Briaud and Tucker (1984-a) indicated that, on the average, the residual point load was about 56% of the "true" ultimate point load, while the residual friction load was about 19% of the "true" ultimate friction load. A thorough examination of these data, however, indicates a very large scatter in the results, so that the average figures introduced poorly represent the data base.

Holloway, et al. (1975), based on theoretical analysis, found that the impact driving system has no effect on residual stress distribution. This means that this distribution is only a function of the pile-soil system.

Hunter and Davisson (1969) back-calculated values of N_q and K_s for both measured and adjusted load distributions. Leonards (1985) also back-calculated N_q values, including the residual stress effect. Many other investigators tried to back-calculate these parameters from load test results to be able to perform the conventional bearing capacity analyses based on limit equilibrium. However, these correlations may not be applicable for soil conditions other than those in which load tests were performed. This is in addition to many other limitations to the conventional bearing capacity analyses. Hence, the designer should exercise caution in using this approach. The best available procedure thus far is probably to use back-calculated parameters obtained from load tests, performed in the design stage, for the specific project at the same site.

Holloway et al., (1975) developed a finite difference solution to the one-dimensional wave equation approximation behavior (after Smith, 1960), coupled with a static equilibrium solution algorithm (after Clough and Duncan, 1971). This better represents the impact pile driving phenomenon of a "complete" hammer blow by approximating static equilibrium in the pile at a point in time when the "useful" work causing pile penetration has been performed. In this manner, subsequent (single) hammer blows may be simulated such that residual pile-soil stresses from the preceding blows are incorporated explicitly. This solution then may be applied at the end of a sequence of hammer blows to simulate an arbitrary axial (static) load test history applied at the pile butt. In this way, the solution utilized the "transfer function" concept described by Vesic (1970-b) presented earlier in this chapter. Predictions, using this program, were made for load test behavior in sites where actual load tests had been performed. Reasonable agreement between the predictions and the actual test results were observed for compression tests, using the nonlinear soil simulation, while the agreement was not as good for uplift tests. Residual stresses, however, were predicted fairly well using this approach. It was found that residual stresses were fully introduced after performing the analysis for five blows. Attempts to correlate the residual loads with the blow count, however, were not successful (Holloway et al., 1978).

Hery (1983) used a similar approach to predict the residual stresses distribution along the pile shaft. He modified the wave equation computer program "WEAP" which was developed by Goble and Rausche (1976) so that it allowed for a multi-blow analysis. The new version of the program was referred to as "CUWEAP". Residual stresses were reasonably predicted by this program, which provides a better prepresentation of the hammer assembly and the soil damping parameters.

Briaud et al., (1983), based on great simplifying assumptions, performed a theoretical analysis to conclude the major factors affecting residual stresses. They assumed that the unloading of the point and friction loads obey a linear elastic model. They also assumed an average constant value of the unit skin friction along the pile shaft, rather than using a reasonable distribution. They performed only a static analysis, which does not simulate the dynamic conditions existing during the pile driving. The analysis takes, as an initial condition, the stress and load distribution in the pile at failure (since soil resistance has already been mobilized by driving the pile). They obtained an expression for the change in pile stress due to unloading (i.e. after driving) which gives the change in pile load at every depth or at the point after unloading (i.e. the residual stress distribution).It is obvious that this analysis is very simplifying, but it was adequate for its purpose. The above mentioned expression showed that the magnitude and distribution of residual loads mainly depend on:

1. Ultimate point and total loads distribution.

2. Pile length.

3. Relative pile-soil stiffness, and unloading characteristics of the pile and soil.

If the pile length is small, skin friction will be small compared to the point load. Similarly, for piles driven through soft layers into a dense layer, the skin friction will be small. In these cases, the residual point load will be small since it can not be higher than the reversed downward friction. As the pile length increases, the downward friction upon unloading would be enough to generate a counterbalancing residual point load plus some upward residual friction.

If a stiff pile is driven into a soft soil, the pile rebound will be small compared to the movement required to reverse friction. Hence the residual point load will be small. On the contrary, if a soft pile is driven into a stiff soil, the pile rebound may be large enough to reverse the friction and hence, generate a large residual point load.

If the unloading curve of unit friction-displacement was stiff, small displacements will be required to reverse the skin friction leading to the development of large residual point load.

Based on their simplified assumptions, Briaud and Tucker (1984-b) found that the effect of the term DΩ is significant (D is the pile length and $\Omega = \sqrt{K'_\tau p/E_p A}$, where K'_τ is the unloading stiffness in friction, E_p is the pile modulus of elasticity, p is the pile perimeter, and A is the cross sectional area of the pile). Hence, an attempt was made to correlate the tip residual stresses and DΩ. As would be expected, the tip residual stresses was found to increase as DΩ increases. Some scatter was observed. Such a correlation can give only an idea, not actual magnitudes, of the residual stresses (as will be discussed later).

Attempts were also made by Briaud and Tucker (1984-b) to correlate the parameters involved with the number of blows obtained from the standard penetration test. A wide scatter, however, was observed. The authors attributed this to the difficulties associated with the standard penetration test and to the natural variation of the soil. By modeling the point resistance-displacement and unit friction-displacement relations using hyperbolas (Briaud et al., 1983), together with the above mentioned correlations with the SPT and a limited data base of actual load test results, Briaud and Tucker (1984-b) developed a simplified design method to predict the load-settlement curve that would have been obtained from a load test. They compared this prediction with actual test results. It should be noticed that these comparisons do not give a true idea of accuracy since they show the precision of the method on the data base used to develop it. It could have been preferable to compare the method with other test results. The comparisons showed, however, that the accuracy of the ultimate load prediction is much greater than that of settlement prediction.

Briaud and Tucker (1984-a) used an approach similar to that used by Holloway et al., (1975) based on one dimensional wave propagation to predict the residual stress distribution. They found that considering the residual stresses in the analysis predicts a smaller number of blows to reach certain capacity for hard driving conditions (i.e. the pile is easier to drive because of the existence of the residual stresses). Some scatter was observed when the predicted residual stresses were compared with those measured in actual load tests, using the available methods summarized earlier (e.g. by Hunter and Davisson, 1969). Briaud and Tucker (1984-a) concluded that a better method for actually measuring the residual stresses in the field, from a load test, should be developed. Correlating the residual stresses with the blow count was also unsuccessful.

4.4 Numerical Evaluation of Residual Stresses and Factors Affecting These Stresses

The study performed by the author included obtaining the residual forces due to driving the piles into cohesionless soils using the wave equation approach. The effect of considering the residual stresses in the analysis is demonstrated together with the main factors that influence the magnitude and distribution of these stresses. This study led to a general technique developed by the author to predict the magnitude and distribution of the residual driving loads below the pile tip and along its shaft. This technique will be demonstrated in a subsequent chapter.

4.4.1 Obtaining Residual Stresses:

A wave equation technique similar to those used by Holloway et al., (1975,1978); Hery (1983); and Briaud et al., (1983,1984) was used to obtain the magnitude and distribution of residual forces below the pile tip and along its shaft. The basic concept of this technique is to interrupt the dynamic phase of the wave equation analysis at a certain time and, using the values at this time, to find the displacement and soil resistances at the end of the blow, that is when the pile is at rest or when a static equilibrium of the system is satisfied (Hery, 1983). Holloway et al., (1975), in the program "DUKFOR," stop the dynamic process when the "useful" work is done, i.e. the sum of all the dashpot forces is less than to a prescribed minimum value (1 kip). Hery (1983) in the program "CUWEAP" interrupts the dynamic analysis when the pile tip starts to rebound, which also indicates that the "useful" penetration work is complete. The output of the dynamic analysis (forces and displacements) is used to achieve static equilibrium of the forces. This is a solution for one hammer blow. All the outcome is then used as initial conditions for a new blow, i.e. another dynamic analysis followed again by the imposition of a static equilibrium. According to Holloway et al., (1975) the residual stress distribution generally remains unchanged after three blows (i.e. displacements and soil resistances will converge within these three blows). At this stage, the set from the last blow would be the same as for any subsequent blow (since forces are the same). Hence, the set can be used as a criterion for checking the convergence. In case of diesel hammers, the stroke is also involved, hence, convergence of the stroke and of the set must be checked simultaneously. Hery (1983) found that the convergence occurred within 2 to 9 blows, the latter figure for a very high ultimate capacity. For common values of the pile load, the convergence criterion was usually satisfied after 2 to 5 iterations. For further details about this approach, the reader may refer to Holloway et al., (1975) and Hery (1983).

The approach used by Holloway et al., (1975) utilized a wave equation analysis similar to the one proposed by Smith (1960) which is good only for drop and steam hammers. Hery (1983) used the wave equation analysis described as the "WEAP" program, which was developed by Goble and Rausche (1976). This approach gave better simulation of the diesel hammers, and is now considered to be one of the best wave equation programs for analyzing pile driving. The version developed by Hery (1983), which included the determination of the residual stresses, was called "CUWEAP". Residual stresses obtained by this program compared quite well with the reported data, both in magnitude and distribution (Hery, 1983). Briaud et al., (1983,1984) used another wave equation model for obtaining residual stresses based upon the same ideas. However, they assumed a uniform shaft friction distribution rather than another reasonable distribution. In one case (pile no. 3, Arkansas River Project), the mean value of the ratio of the predicted residual load over the measured point load was only 0.53, with a coefficient of variation of 0.669 (Briaud et al., 1983). This constitutes a fairly poor prediction of the residual tip load.

The above mentioned discussion lead to the conclusion that the "CUWEAP" program is probably the best wave equation computer program that can be used for the residual forces evaluation. Hence, it was decided to use it to perform the analysis for the current study. Table (15) from Hery (1983) summarizes the algo-

Table (15) Residual Stress Analysis Algorithm (from Hery, 1983)

Step	Comment	Subroutine
1	Store the initial conditions during the dynamic analysis.	STORE
2	Assemble the stiffness matrix and store it in a two column array.	SPLEEN
3	Solve the linear system.	SOLVE
4	Calculate and check soil resistance. Modify the equations and go back to step 3 if necessary.	RSOLVE
5	Compute the pile forces.	SPLEEN
6	Check convergence of the overall cycle. If there is no convergence, starts another cycle with new initial conditions at step 1.	AIRSTM(A/S hammers) WEAP(Diesel Hammer)
7	Print results of the residual stress analysis. If several pile capacities are to be analyzed, start a new analysis at step 1. When all capacities are analyzed, print summary.	RSOUT 1 WEAP RSOUT2

rithm used in "CUWEAP" to perform the residual stress analysis.

4.4.2 Effect of Considering Residual Stresses in the Dynamic Analysis:

The most direct effect of including the residual stresses is the change in the blow count-total resistance relationship obtained from the wave equation analysis. Figure (47) shows an example of such relationship with and without including residual stresses. It indicates that the pile is easier to drive if residual stresses are included (i.e. a certain resistance can be achieved with smaller number of blows). This is logical since the accumulation of compressive residual stresses below the pile tip as the driving proceeds facilitates the driving process. In the conventional wave equation analysis, the resistance of the soil spring returns to zero at the end of one blow, while, it does not return to zero if residual stresses are considered (because of the existence of some residual forces). This means that the set should be larger if residual stresses are considered (i.e. the number of blows/ft. is smaller). This is illustrated by the simple diagram shown in Figure (48) from Hery (1983).

The reduction in the number of blows/ft. due to residual stress consideration is directly related to the magnitude of these stresses. An attempt was made to correlate this reduction, expressed as a factor "α_N" (which is the ratio between the number of blows/ft. with and without considering residual stresses) with

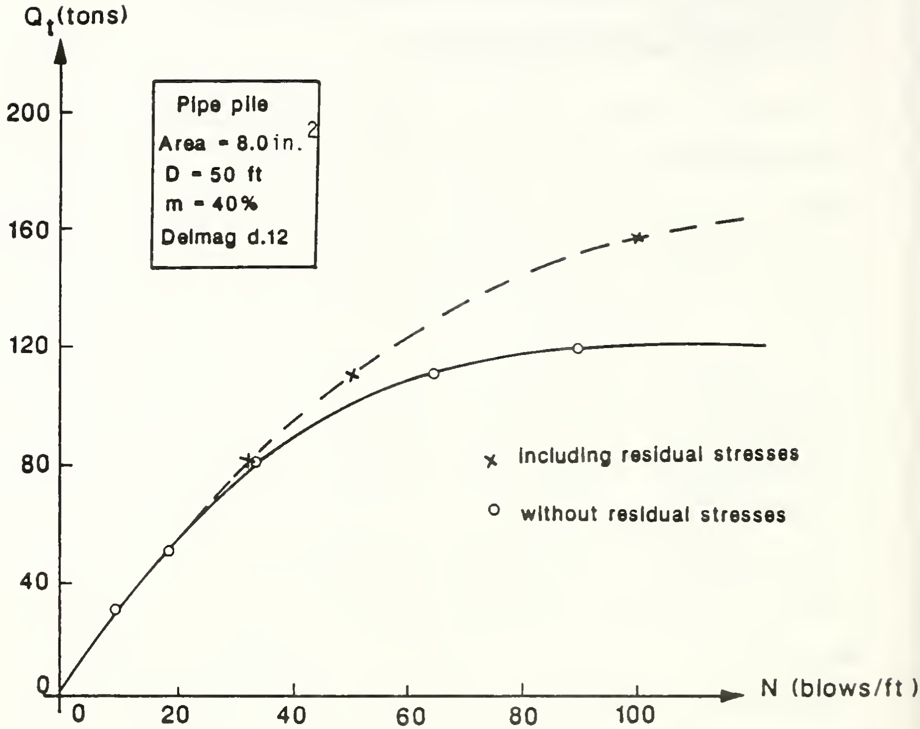


Fig. 47. Effect of Including Residual Stresses on the Number of Blows/ft.

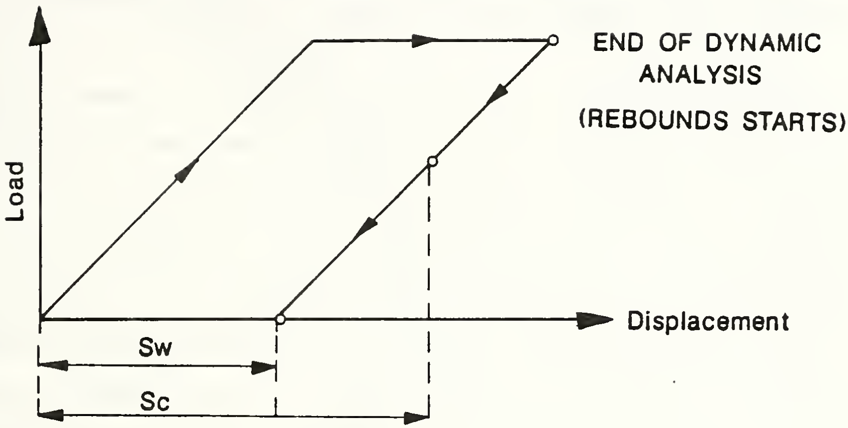


Fig. 48. Determination of the set in WEAP (S_w) and CUWEAP (S_c) (from Hery, 1983)

the residual stress percent "a_r" which is defined as:

$$a_r = \frac{Q_{PR}}{Q_P} \times 100 \qquad (4.1)$$

where Q_P is the load carried by the pile point and Q_{PR} is the residual load calculated at the point.

Examples of such relationship are shown in Figure (49). This figure indicates that low values of residual forces have no effect on the number of blows. As the residual stresses increase, the factor "α_N" decreases with an increasing rate. At high values of "a_r," "α_N" becomes very small. This is due to the fact that refusal may be indicated if residual stresses are not considered, giving a very high number of blows, while the actual condition with the existence of residual stresses does not indicate such refusal (as indicated in Figure 47). Different curves like the ones shown in Figure (49) are expected for different pile-soil-driving system conditions. A large number of data points obtained from Hery (1983), Briaud et al., (1984) and from results of computer runs made during the course of this study, are shown in Figure (50). The curves connecting the points for each case are not shown for better illustration. Some scatter can be observed due the differences between the conditions associated with each case. Upper and lower bounds, together with the mean curve for "α_N" are also shown. In spite of this scatter, the mean curve can be used to give a reasonable estimate of "α_N",

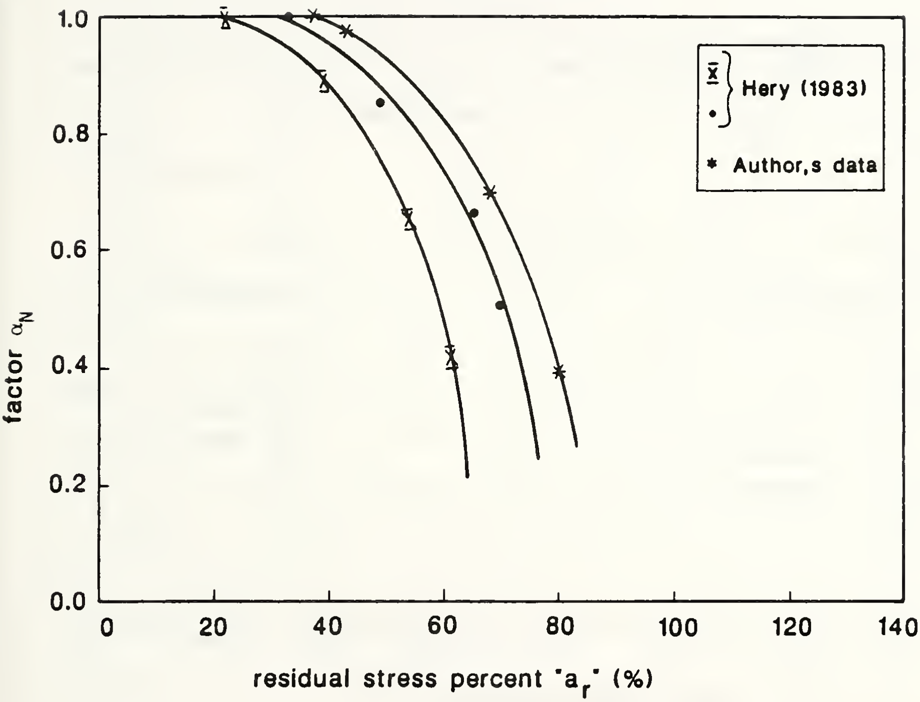


Fig. 49. Represnetative Data for the Factor "α_N"

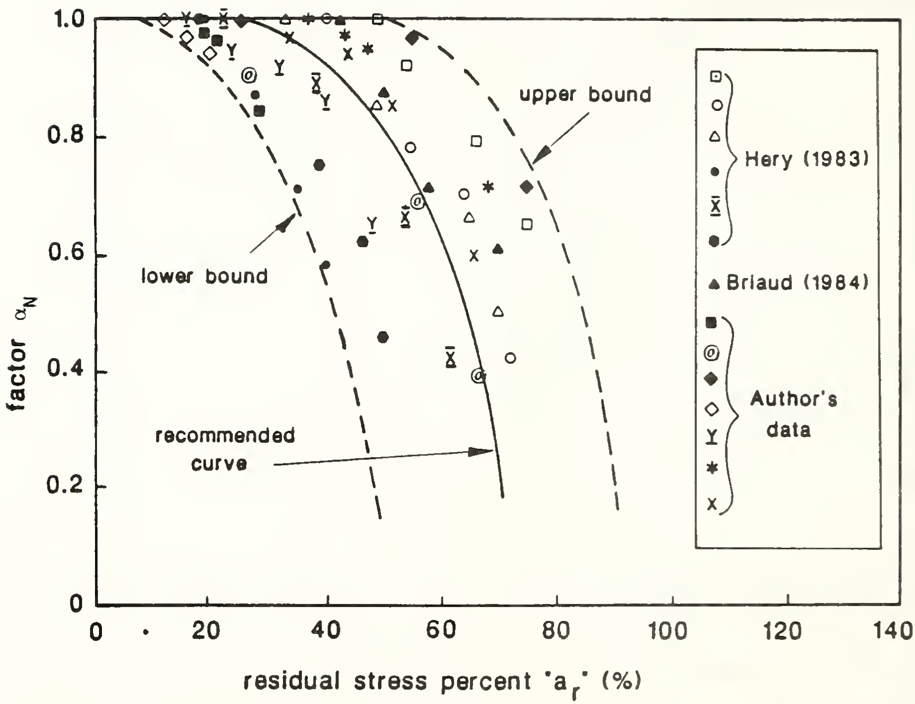


Fig. 50. Upper and Lower Bounds and Recommended Values for α_N

provided that "a_r" is known. Prediction of "a_r" will be described later in this chapter. Hence, if a simple wave equation analysis is performed, without considering residual stresses, and if "a_r" can be predicted, a reasonable estimate of the actual expected number of blows can be made (multiplying the number of blows obtained from the simple wave equation analysis by "α_N"), and a more reasonable N-resistance relationship can be obtained.

Another effect of the residual stresses is the change in the maximum driving stresses. Wave equation analyses performed by Hery (1983) indicate that the driving stresses are slightly higher if residual stresses are considered, which means that not considering residual stresses in the anlysis will lead to underestimating the maximum driving stresses. This was confirmed by a number of computer runs during the course of this study. Again, a correlation of the increase in driving stresses due to considering residual forces, expressed as a factor α_s, was attempted with the residual stress percent "a_r". Figure (51) shows some data points for such a relationship. Values of α_s ranged between 1.0 (no change in stresses) to about 1.14 (stresses increase by 14% if residual stresses were considered). The average value of "α_s" seemed to be constant irrespective of the value of "a_r", for values of a_r greater than about 10%. This average value was about 1.05. However, to be on the safe side, one may assume that the compressive driving stresses are about

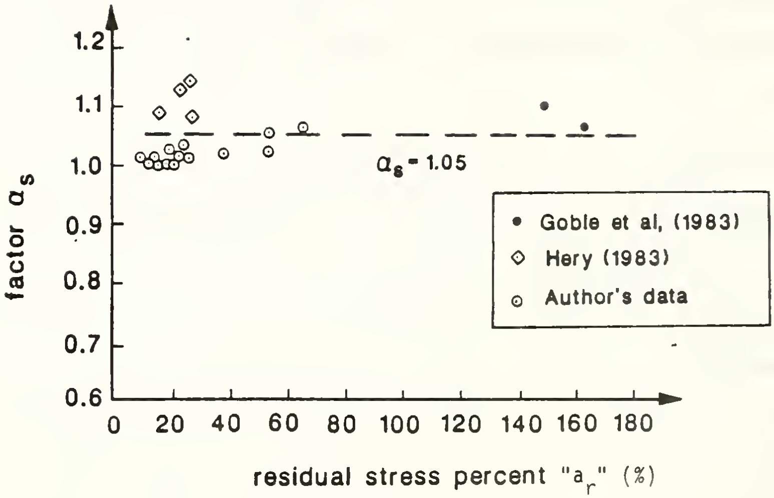


Fig. 51. Effect of Including Residual Stresses on Pile Driving Stresses

10% higher than the values obtained from a simple wave equation analysis that does not take residual stresses into account.

4.4.3 Distribution of Residual Forces along Pile Shaft:

The magnitude of the residual forces after driving can be obtained using the wave equation analysis outlined before. Figure (52) shows both the residual skin friction distribution and the residual pile load distribution for one of the cases studied. Curve (a) gives the residual load at every pile depth from ground surface down to this particular depth, while curve (b) gives the unit skin friction along the pile shaft multiplied by its perimeter. It should be noticed that curve (b) resembles the first derivative, or the slope, of curve (a), as explained by Vesic (1977) (notice the correspondence of points (1) and (2) on both curves). It was found that the general shapes of these curves are typical for all the residual stress analyses made by "CUWEAP," irrespective of the variables involved in the problem. These shapes agree with the ones obtained by the Hunter-Davisson procedure (although the magnitudes are somewhat different, as will be explained later), and with the direct measurements made by Gregersen et al., (1973). They also agree with the shapes obtained by the "DUKFOR" program used by Holloway et al., (1975). They indicate the existence of a negative friction for the upper portion of the pile, down to a critical depth "Z_{cr}" below which there is an upward residual friction load in addition to the residual tip load. The sum of the upward residual tip load,

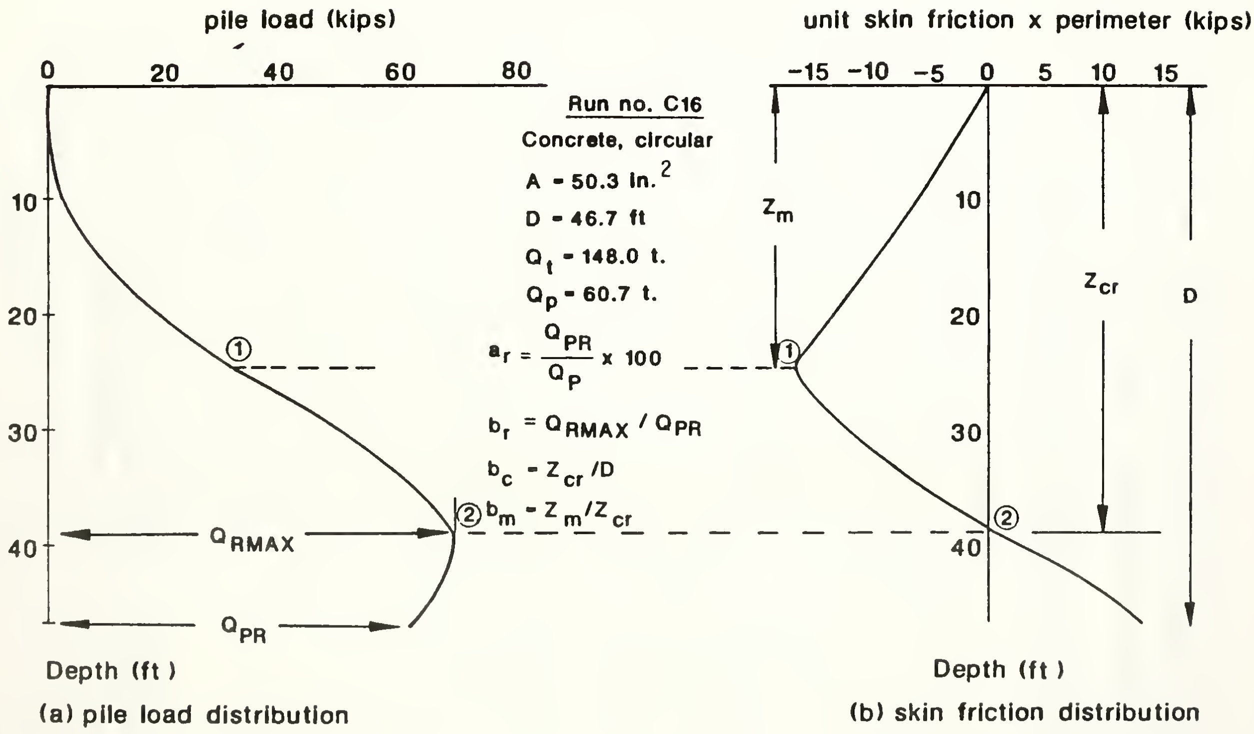


Fig. 52. Typical Residual Pile Load and Skin Friction Distribution

upward residual friction, and downward negative friction is zero, since the pile is in equilibrium, as explained earlier.

In order to define the shapes of curves (a) and (b) one needs to know the residual point load "Q_{PR}", the maximum residual shaft load "Q_{RMAX}" that corresponds to the depth "Z_{cr}", and the depths "Z_{cr}" and "Z_m" shown on Figure (52). If the residual stress percent "a_r" is known (as will be shown later) the residual point load "Q_{PR}" can be estimated. Three ratios, "b_r", "b_c", and "b_m", can be defined for the description of these curves, such that:

$$b_r = Q_{RMAX}/Q_{PR} \tag{4.2}$$

$$b_c = Z_{cr}/D \tag{4.3}$$

and

$$b_m = Z_m/Z_{cr} \tag{4.4}$$

in which "D" is the pile length.

A large number of computer runs indicated that the ratio "b_r" for sands is almost constant, with an average value of 1.05 and a coefficient of variation of about 1.5%. The same applies to the ratio "b_m" which showed an average value of 0.60 and a coefficient of variation of about 10%. It should be noted that these values apply only for piles driven into cohesionless soils, for which this study was performed.

The ratio "b_c", which defines the critical depth at which the residual friction is reversed from negative to positive, showed greater variability. It was found that the main factors that govern this ratio are the pile length, the amount of pile load that is transferred to the soil by friction, expressed in terms of a parameter "m" which is defined as the percentage of load carried by skin friction, and the total ultimate soil resistance "Q_t". For certain values of "D" and "m", however, the ratio "b_c" varied little with "Q_t". This is indicated by part of the data that are shown in Table (16). Figure (53) gives the data that describe the effect of pile length and skin friction percent on "b_c". If "m" were fixed to a value of 40%, the data points that describe the relation between pile length and "b_c" are shown on part (a) of Figure (53). A decreasing trend of "b_c" with increasing "D" can be observed, which means that as the pile gets longer, the critical depth "Z_{cr}" relative to the pile length gets smaller. This can be explained by the fact that as the pile gets longer, there will be a greater amount of downward negative friction that needs to be balanced by a residual point load and an upward friction, which means that the length of the portion in which there is an upward friction along the pile shaft should increase. On the other hand, if the pile length were fixed, as shown on part (b) of Figure (53), the critical depth ratio "b_c" should decrease as the skin friction percent "m" increases. The

Table (16) Effect of Total Soil Resistance on Critical Depth Ratio b_c

run no.	pile section	length (ft)	m (%)	Q_t (tons)	b_c
72	concrete, d = 8 in.	40	40	70	0.88
73	"	"	"	60	0.88
78	concrete, d = 10 in.	"	"	100	0.89
79	"	"	"	75	0.87
82	concrete, d = 12 in.	60	"	160	0.88
81	"	"	"	130	0.88
84	"	"	"	120	0.87
95	concrete, d = 16 in.	80	"	270	0.86
102	"	"	"	180	0.85
S43	steel, A = 20 $in.^2$	100	"	80	0.875
S44	"	"	"	140	0.88
S45	"	"	"	200	0.90
S26	steel, A = 15 $in.^2$	75	"	60	0.86
S27	"	"	"	100	0.87

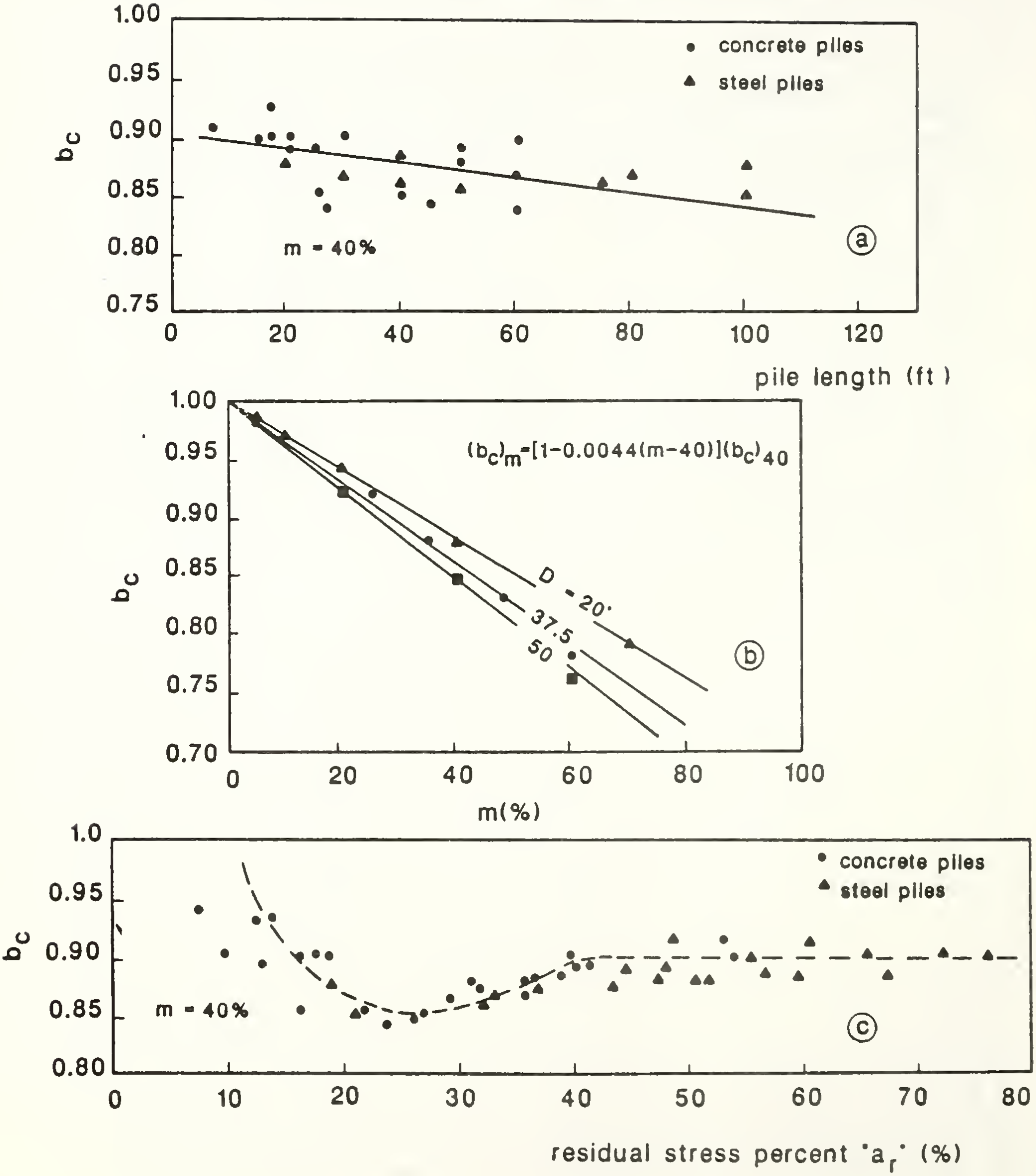


Fig. 53. Prediction of the Critical Depth Ratio for Sands

same explanation given above can also be used to explain this trend. The following correlation between "b_c" and "m" was derived for a certain constant pile length:

$$(b_c)_m = [1.0 - 0.0044\ (m - 40)]\ (b_c)_{40} \tag{4.5}$$

This relation gives the ratio "b_c" at any value of "m" as a function of the ratio "b_c" at m = 40%. Hence, one can enter with the pile length into part (a) of Figure (53) to obtain "b_c" at m = 40%. Then Equation (4.5) can be used to get the correct value of "b_c" for the estimated value of "m". The ratio "b_c" for piles driven into cohesionless soil ranges between about 0.70 and 1.00, according to the variables of the problem, as shown in Figure (53).

As an example for the prediction of "b_c", the data reported by Gregersen et al., (1973) for pile number (D/A), which was a circular prestressed concrete pile having a 0.92 ft diameter and 52.5 ft length, were examined. According to Figure (53.a), "b_c" for 40% skin friction equals 0.87. Using Equation (4.5) with m = 76.5% indicates that the predicted value of "b_c" is 0.73. Measurements reported by Gregersen et al., for this pile indicated a value for "b_c" of 0.70. This example shows that the suggested prediction is quite good. Unfortunately, the data reported by Gregersen et al., (1973) were the only data that could be obtained in which there were reliable actual measurements of the residual loads along the pile shaft.

4.4.4 Factors Affecting the Magnitude of Residual Stresses:

In order to develop a procedure for predicting the magnitude of residual stresses, a parametric study was conducted, using "CUWEAP", to examine the effect of the various parameters involved. The factors involved in this study were: the driving system, including the types of hammers, cushions, ... etc.; the total soil resistance; the percentage of load transferred by shaft friction; and the pile material, cross section and length. This study was conducted for piles driven into cohesionless soils only. The dependent variable that was studied was the residual stress percent "a_r", which indicates the amount of residual load below the pile tip due to driving.

4.4.4.1 Effect of the Driving System and Elements:

Three hammer types were used in this analysis, giving a wide range of energies delivered to the pile during driving, with other variables being the same. Table (17) summarized the results of this analysis. It indicates that the point residual force slightly increases as the hammer rated energy increases. The range of "a_r", however, that corresponds to the wide range of hammer energies was quite narrow.

When the hammer used was fixed, and the driving elements, like caps and cushions, are changed, the results given in Table

Table (17) Effect of Hammer Type and Energy on the Residual Stress Percent*

Run No.	Hammer Model	Manufacturer	Maximum Energy Rating (kips. ft.)	Power Type	Ram Weight (kips)	Residual Stress Percent a (%)
C68	VULC.2	Vulcan	7.3	Single acting air/steam hammer	3.00	24.2
C56	DE1.D-12	Delmag	22.5	Diesel, continuous fuel pump	2.75	27.4
C67	KOBEK-35	Kobe	70.8	Diesel, continuous fuel pump	7.72	28.5

* All piles used were concrete piles with circular x-sec.; 10.1 in. diameter, 37.5 ft long; total soil resistance = 125.0 t., 25% of which is provided by shaft friction; 12 x 12 in. Combest cap, no cushion.

(18) were obtained. These results indicate that the type of cap or cushion used in the driving procedure has almost no effect on the magnitude of the tip residual stresses.

The above mentioned results indicate that the residual stress phenomenon is mainly affected by the pile-soil combination rather than by the driving system and elements. This conclusion was expected, since the residual stresses are caused by the load/rebound cycles occurring during the driving process, as explained earlier. Hence, they should mainly be affected by the relative pile/soil stiffness, rather than by the driving elements. The same conclusion was also reached by Holloway et al., (1975) using the computer program "DUKFOR". This result makes the prediction of the magnitude of residual stresses somewhat simpler, since it may be assumed that they are not affected by the driving system used.

4.4.4.2. Effect of Total Soil Resistance:

One of the major parameters that is expected to influence the magnitude of the residual forces is the total resistance provided by the soil. Figure (54))gives some variations of the residual stress percent with the total pile capacity, provided that all other parameters are the same for each curve. As the total capacity increases, the point residual stress increases because of the pile elastic rebound, and hence the negative friction that is balanced by the upward residual forces increases. The

Table (18) Effect of Types of Caps and Cushions on the Residual Stress Percent*

Run No.	Types of Caps and Cushions	Residual Stress Percent (%)[a]
C56	12 x 12 in. Conbest cap, weight = 0.95 kips, stiffness = 21,000 kips/in. - no cushion.	27.4
C69	16 x 16 in. oak cap, weight = 0.91 kips, stiffness = 28,000 kips/in. - no cushion	27.8
C70	12 x 12 in. Conbest cap, weight = 0.95 kips, stiffness = 21,000 kips/in. - Micarta + aluminum cushion, E = 700 ksi, coefficient of restitution = 0.80	26.75

* All piles used were concrete piles, with circular x-sec.; 10.0 in. diameter, 37.5 ft long; total soil resistance = 125.0 t., 25% of which is provided by shaft friction. All driven by a DEL.D-12 hammer.

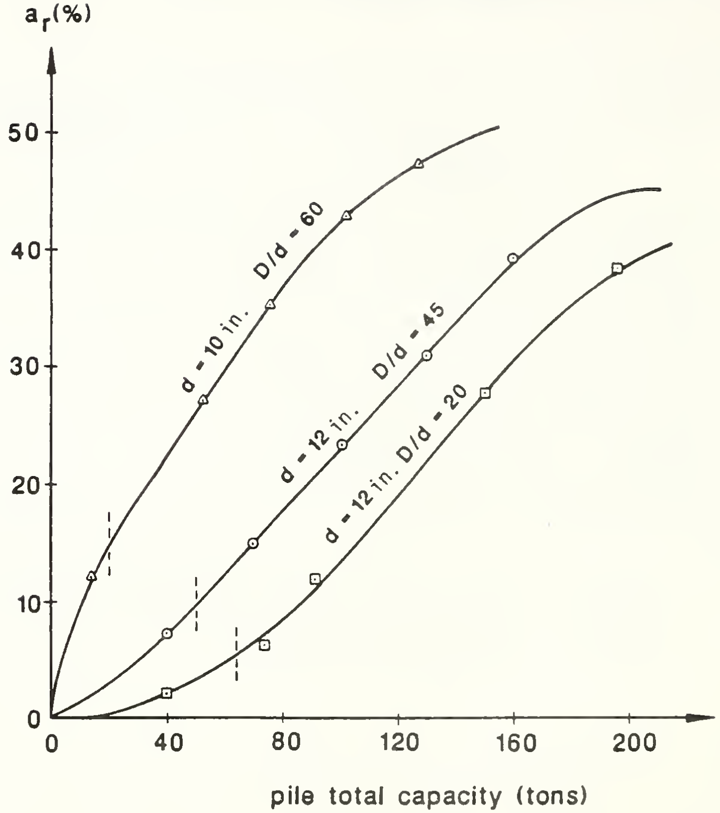


Fig. 54. Effect of Total Pile Capacity on the Residual Stress Percent "a_r"

variation is almost linear except at the early and late stages of the curve. Similar results were also reported by Hery (1983). For the high values of pile capacity, the strength of the pile material may be exceeded, which makes the curves become flatter. On the other hand, the amount of pile rebound should mobilize some negative friction up to the point where no more friction can be mobilized, which limits the magnitude of the residual stresses, giving the tendency of the curve to be flatter at higher values of the pile capacity. Very low values of ultimate pile capacities shown on these curves are usually not realistic for piles driven into cohesionless soils. Hence, the portion of the curve which begins when it becomes linear can be used in a prediction procedure as will be shown later.

4.4.4.3. Effect of Skin Friction Percent:

One of the major factors that is expected to have an important effect on the magnitude of the tip residual stresses is the amount of load transferred to the soil by shaft friction, relative to the total pile capacity. This can be expressed in terms of the skin friction percent "m" which is defined by:

$$m = \frac{Q_s}{Q_t} \times 100 \qquad (4.6)$$

where Q_s and Q_t are the shaft and total pile capacities, respectively. The value of Q_s is the "true" shaft friction, without including any residual stresses, which is used as an input in the wave equation analysis.

As a matter of fact, there is an interaction between the mechanism of load transfer and the magnitude of driving residual stresses. In other words, the residual stresses are influenced by the nature of load transfer, while this nature changes due to the existence of the residual forces. Figures (55) and (56) show the relation between the skin friction percent "m" and the residual stress percent "a_r" for selected concrete and steel piles, respectively. As these curves indicate, "a_r" increases as the value of "m" increases, for the same total pile capacity. Since the amount of residual tip load is a function of the positive/negative skin friction cycles induced by pile driving and rebound, it should be expected to increase as the amount of load transferred by friction increases (larger amount of negative friction will need to be balanced by some upward residual tip load). If no load was transferred by friction, no residual stresses should occur. As "m" increases, "a_r" increases, but the rate of increase is lower after a value of "m" of about 20%. For high values of "m", the curves should become flatter to approach an asymptotic value for m = 100%. Any of the curves shown in Figures (55) and (56) can be approximated by two straight lines of different slopes before and after m $\simeq$ 20%, up to a value of m of about 65%, when they tend to flatten.

As stated earlier, the value of "m" is also affected by "a_r", since the residual stresses tend to increase the shaft

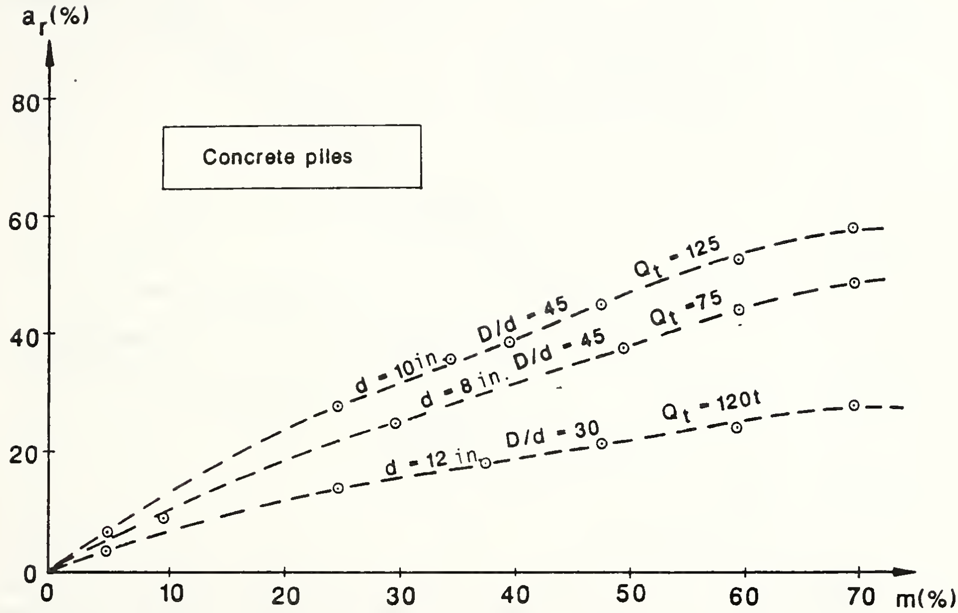


Fig. 55. Effect of the Skin Friction Percent "m" on the Residual Stress Percent "a_r" for Concrete Piles

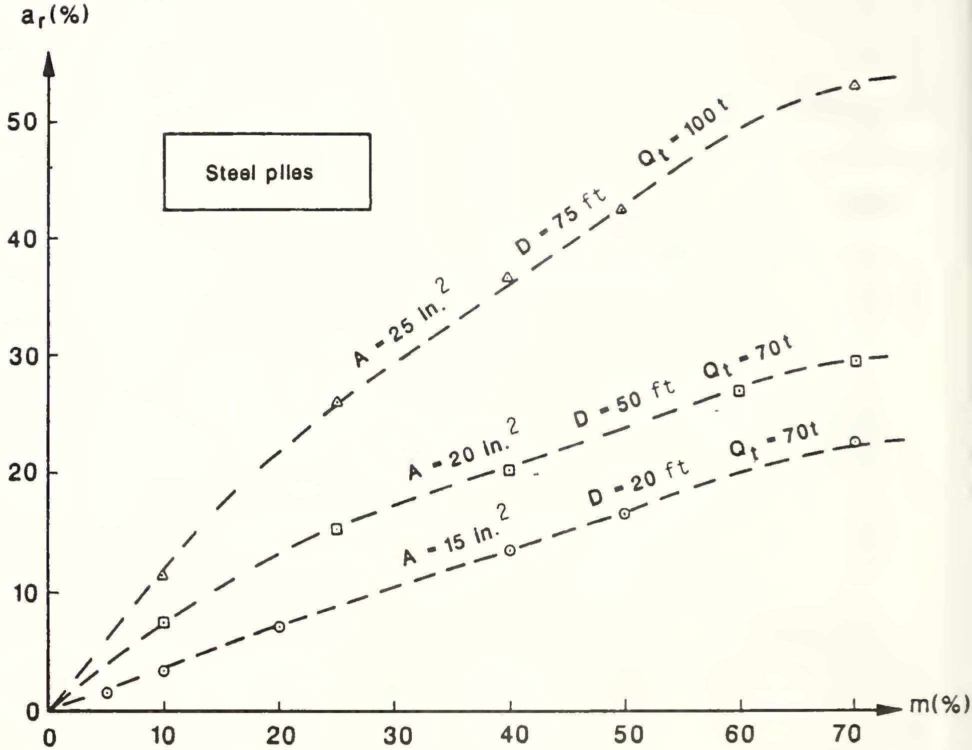


Fig. 56. Effect of the Skin Friction Percent "m" on the Residual Stress Percent "a_r" for Steel Piles

forces and decrease the tip forces that are measured in an axial load test. Therefore, any attempt to predict the value of "a_r", which includes the effect of "m", should be iterative, since m is changed as a_r changes. This factor was not included in any other procedure that involved the prediction of the tip residual stresses (e.g. Briaud and Tucker, 1984-b), although it is of crucial importance. However, it is one of the main factors that is considered in the prediction procedure suggested by this current study.

4.4.4.4. Effect of Pile Length:

Another major parameter which influences the magnitude of the tip residual stress is the length of the pile. Figure (57) shows the relation between the pile length, relative to its diameter, and the residual stress percent "a_r" for selected concrete piles. Also Figure (58) shows the same relation, with the pile length as the independent variable, for selected steel piles. A semi-log scale was used in drawing these curves. Different combinations of pile total capacities and cross sections were used. It is shown that the tip residual stresses increases as the pile length increases, with other conditions remaining constant. For longer piles, the shaft friction forces are higher, and the amount of negative friction resulting from the pile rebound after each blow, which needs to be balanced by an upward residual force for equilibrium, is greater.

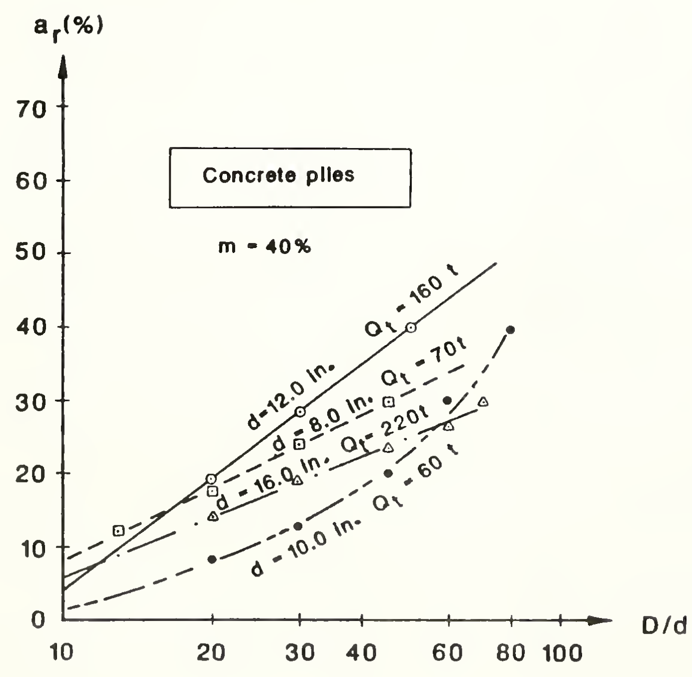


Fig. 57. Effect of Pile Length on Residual Stresses - Concrete Piles

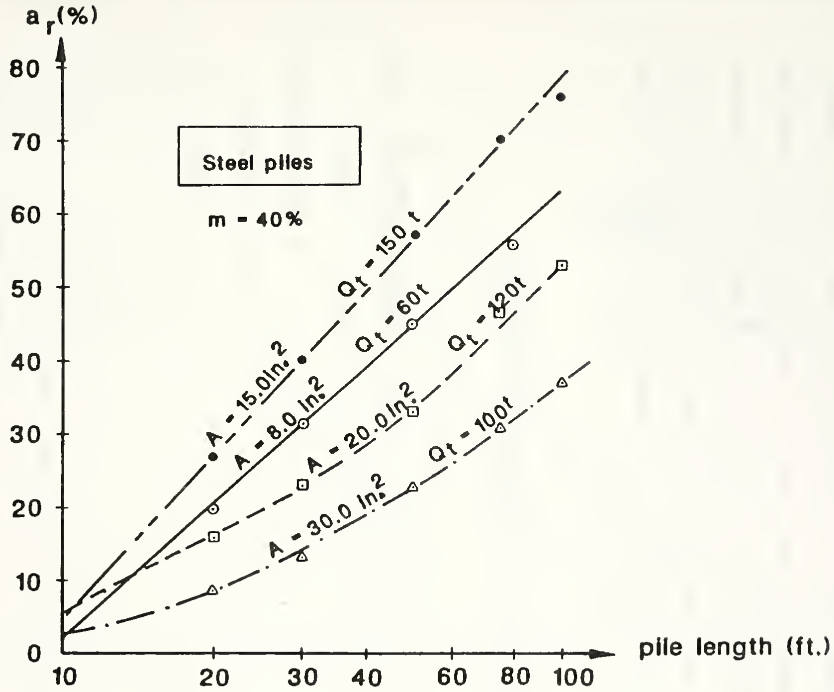


Fig. 58. Effect of Pile Length on Residual Stresses - Steel Piles

As is shown in Figures (57) and (58), the variation of the logarithm of the pile length with the percentage "a_r" is sometimes linear and sometimes nonlinear, depending on the combination of the other parameters that affect the residual stresses. For this reason, it may not be practical to use the pile length as the main independent variable for developing charts to predict "a_r", as will be shown later.

4.4.4.5. Effect of the Pile Cross Sectional Area:

The pile cross sectional area is another major factor that affects the magnitude of the tip residual stresses. Figure (59) shows the relation between the pile cross sectional area, or pile diameter, and the residual stress percent "a_r" for different concrete and steel piles. These curves indicate that, for other parameters remaining constant, the residual stress percent "a_r" increases exponentialy as the pile cross section becomes smaller. For the same soil conditions, as the pile section gets smaller, the relative pile/soil stiffness gets smaller, which should directly result in higher tip residual forces. The exponential increase of "a_r" with decreasing the cross section indicates that the residual stresses phenomenon is more important for the more flexible piles. This observation was confirmed by certain test cases with Monotube piles reported by Goble and Hauge (1978) for the Union Metal Manufacturing Company. The actual field driving data were much in disagreement with the predictions made using the computer program "WEAP", which does not take the residual

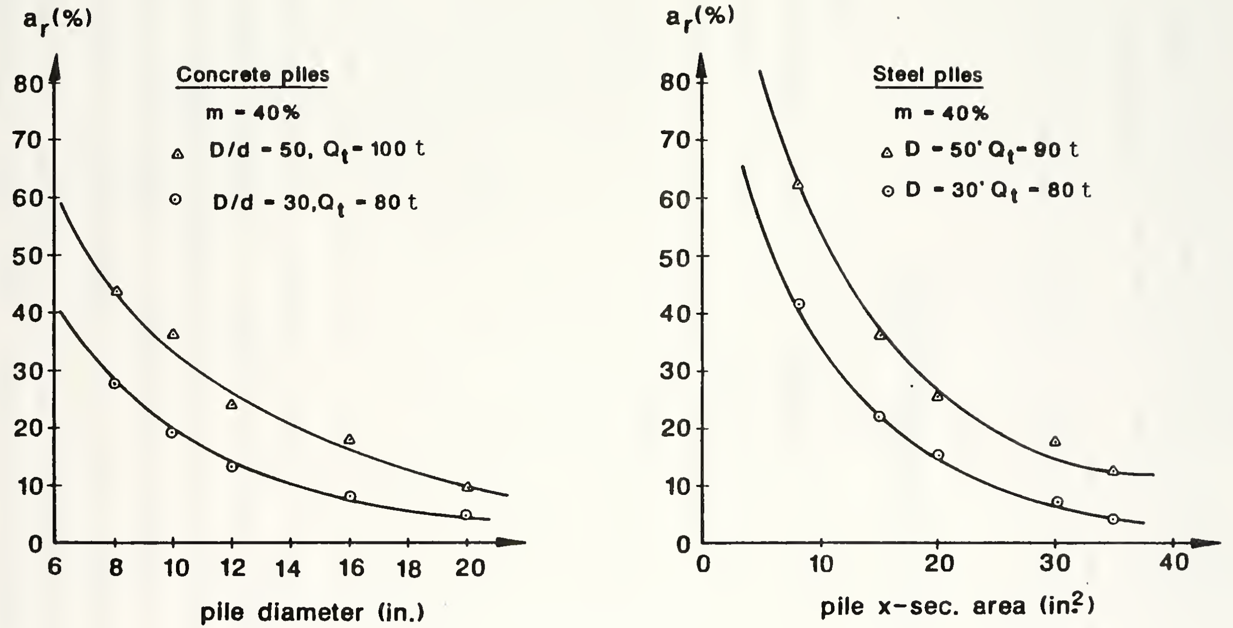


Fig. 59. Effect of Pile Cross Section Area on the Residual Stress Percent "a_r"

stresses into account. Because the Monotube piles are rather flexible, the discrepancies might be explained in terms of neglecting residual stresses. Further analysis performed by Hery (1983) for the same piles, including residual stresses, showed that the predictions were much closer to the actual field measurements.

4.4.4.6. Effect of the Pile Material:

The pile material is expected to have a great influence on the residual stresses phenomenon. As the pile becomes stiffer, residual forces should decrease, for the same soil conditions. In order to examine the effect of this factor, two piles were assumed, each having a length of 40.0 ft, cross-sectional area of 20.0 in.^{2}, driven to a total capacity of 60.0 tons, 40% of which is transferred by shaft friction. One of the piles was a concrete pile with E = 5,000 ksi, while the other was a steel pile with E = 30,000 ksi. The residual stress percent "a_r" obtained by the wave equation analysis for the concrete pile was 66.1%, while it was only 14.6% for the steel pile. This means that increasing the pile stiffness by about 6 times resulted in a decrease of "a_r" of about 4.5 times. It should be noted, however, that this does not mean that residual stresses are always much higher for concrete piles than for steel piles, since the cross sectional area used in this example was quite small for the practical range of diameters usually used for concrete piles. The example demonstrates, however, that predicting residual stresses for different pile materials is quite different. Different charts should be used for different pile materials, as will be shown later.

For the same pile material, the value of "E" also makes some difference. To examine this effect, another concrete pile was considered, having the same characteristics as the ones used in the previous example; except that it was made of a poorer concrete that results in a value of "E" of 3,500 ksi. The value of "a_r" obtained was about 10% higher than that for the other concrete pile that had a higher value of E. This should also be taken into account in any prediction procedure.

It should be noted that the curves and relations shown in the above mentioned sections, regarding the factors affecting residual stresses, were obtained assuming that the pile was driven into cohesionless soils, a principal restraint for this study. For piles driven into cohesive soils, however, the general trends may not be different. Certain parameters are different in the latter case, e.g. the shaft friction distribution in cohesive soils. Briaud et al., (1983) stated that little or no residual stresses are expected for piles driven in clay. One can not, however, generalize this statment. This may be true for piles driven through weak cohesive soils into dense sand or hard clay strata, in which most of the load is transferred by point bearing. This means that very small residual stresses are expected (low values of "m", as explained earlier). In some other cases, however, significant residual stresses can exist for piles entirely driven in clay. This is particularly the case of long friction piles driven through soft clay layers. This is because the stiffness of such piles is usually low (long pile, small cross section) and most of the load is transferred to the soil by shaft friction. The significance of this observation was briefly examined by performing a "CUWEAP" analysis for a steel pipe pile of 10.0 in. diameter and 0.19 in. wall thickness. Its length was 100 ft, penetrating soft clay layers. It was assumed that about 90% of the total load would be transmitted to the soil by shaft friction. The analysis gave values of residual point loads as high as one and half times the assumed point load (i.e., about 15% of the total pile capacity). This is contrary to what was stated by Briaud et al., (1983), viz., that these stresses can not exceed 5% of the total load for clays. It should also be noted that the parameters describing the shape of the residual stress distribution for cohesive soils may be somewhat different from these obtained for cohesionless soils (e.g. the critical depth relative to the pile length, etc.).

4.5 The Development of an Approximate Technique for Residual Stresses Prediction

The foregoing discussions illustrated the importance of considering the residual stresses due to driving in the pile foundation analysis. The prediction of these stresses is still beyond the current state-of-the-art. Although the direct measurement of these forces is the most accurate technique for their evaluation, many difficulties and uncertainties still prevent the engineer from measuring accurate values. Other load test sequences that have been recommended for the evaluation of these forces imply invalid assumptions which leads to an incorrect evaluation of the residual stresses. On the other hand, field determination of these stresses require pile load tests in the design stage, which is still not very common in every day practice. Certain predictions were made using the wave equation analyses which have proven to be quite satisfactory. This analysis requires the availability of a good wave equation program, plus computer hardware. The engineer, however, may need a simple technique to make quick estimates and acquire good first approximations, besides the fact that the required computer software and hardware may not be available. Techniques have been recommended to make such predictions. They make, however, too many simplifying assumptions. Therefore, the author of this study felt the need for such a prediction procedure and tried to use the wave equation analysis for the development of this technique, since it was proven to be the best way available for residual stress evaluation. In this chapter, the disadvantages of the other techniques that have been proposed are discussed. A recommended procedure is described and illustrated by certain numerical examples. Comparisons of these predictions to reported actual measurements, as well as the other prediction techniques, are reported. One of the main purposes of this prediction is to facilitate better interpretation of load test results, especially for these which are used to develop empirical approaches for pile capacity predictions.

4.5.1 The Hunter-Davisson Method:

The Hunter-Davisson method for obtaining residual stresses was described in detail in a previous chapter. It requires a certain sequence for pile testing which involves performing a pulling test after the main compression test. One major assumption of this procedure is that it assumes that tension loading results in no residual stresses. Actual measurements, however, made by Goble et al., (1972) indicated the existence of some residual stresses just after the pulling test, which means that some residual loads do exist due to the pulling test only. Simplified approximate analyses performed by Briaud et al., (1983) and by Briaud and Tucker (1984-b) also indicated that this assumption is incorrect. This analysis, however, showed that no significant residual stresses can result after a pulling test if the pile is not very long. The error is zero at the top and bottom of the pile and peaks towards the middle of the pile, where the residual tension load can be as high as 25% of the ultimate tension load for very long piles according to theory (Briaud and Tucker, 1984-b).

Another implicit assumption of this procedure, although not realized by Hunter and Davisson (1969), is that the distribution of shaft friction in loading is the same as in the case of unloading. This assumption enabled them to obtain the residual load distribution from unloading the pile, despite the fact that they originally resulted from loading (either from the driving of the pile or from the compression testing). Load test data, however, do not support this implicit assumption. For example, the interpretation of test results reported by Vesic (1970), which was made by the author of this study, indicates that the unit friction distribution is totally different for the unloading case (Figure (60)). Leonards and Lovell (1979) stated that loading and unloading in compression followed by loading and unloading in tension can result in considerable changes from the initial conditions, especially in the distribution of shaft friction.

The above discussion shows that the Hunter-Davisson procedure can be used with fair success in predicting the residual tip stresses, but it results in an incorrect evaluation of the distribution of shaft residual stresses along the pile length.

4.5.2 <u>The Holloway Procedure:</u>

Holloway et al., (1975) used the wave equation computer program "DUKFOR" to predict the residual stresses, as described earlier. The sequence of analysis was quite reasonable, the wave equation solution algorithm, however, was not rigorous enough to include such factors as the diesel hammers modeling, the more accurate soil damping inclusion, etc., as was done later in the programs "WEAP" (Goble and Rausche, 1976) and "CUWEAP" (Hery,

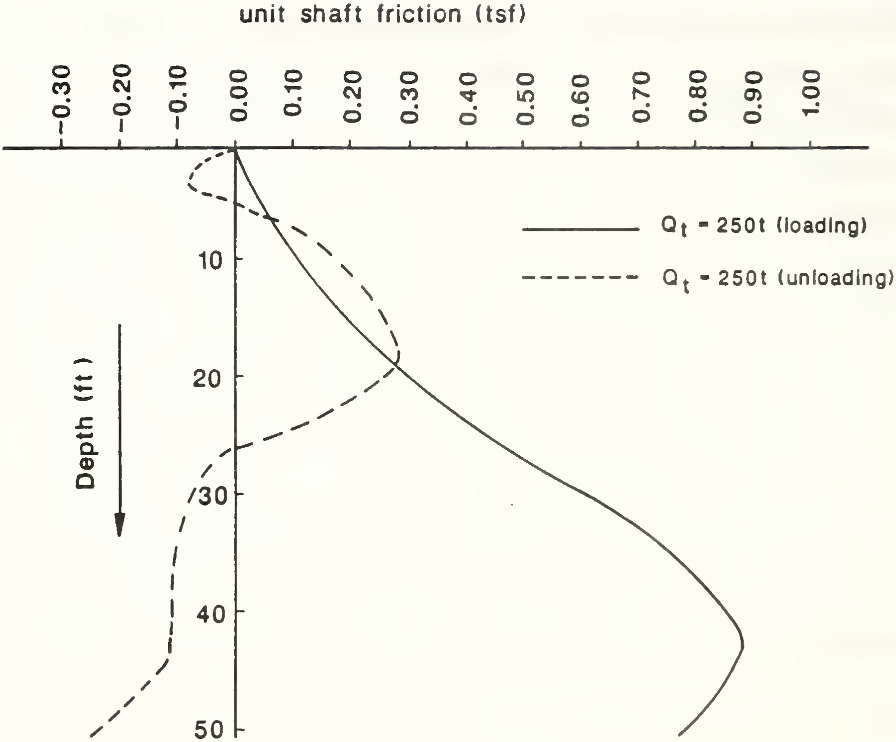


Fig. 60. Unit Shaft Friction Distribution in Loading and Unloading, Interpretation of Results Reported by Vesic, 1970, for Pile No. H-15

1983). These deficiencies in the model could result in inaccurate predictions. The main ideas described by Holloway, however, were the basis for all the prediction procedures that used the wave equation analysis to predict the residual stresses.

4.5.3 The Briaud-Tucker Procedure:

Briaud and Tucker (1984-b), based on a very approximate analysis, concluded that the point residual stresses could be correlated with a certain parameter Ω, defined by:

$$\Omega = \sqrt{\frac{k_\tau \, p}{A_p \, E_p}} \tag{4.7}$$

where:

k_τ = loading stiffness for skin friction

p = pile perimeter

A_p = area of pile tip

E_p = pile modulus of elasticity

The value of k_τ was obtained by the following correlation:

$$k_\tau = 5.01\ (N_{side})^{0.27} \quad \text{tsf/in.} \tag{4.8}$$

where N_{side} is the average SPT blow count within the shaft length considered, without any of the corrections that have been proposed for the SPT number of blows. The data from which Briaud and Tucker (1984-b) formed this correlation are shown on Figure

(61). The residual point stress, q_{res}, can be estimated by:

$$q_{res} = 5.57\ D\ \Omega \quad \text{tsf} \tag{4.9}$$

where D is the pile length in feet.

Several shortcomings of this procedure may be shown. First, it is assumed that the point residual stresses are related only to the parameter Ω and the pile length. An analysis performed and described earlier in this study showed that two other major parameters were not taken into account, namely the fraction of load transfered into the soil by skin friction, and the magnitude of the total pile capacity. It was shown that these two parameters are among the controlling factors that govern the magnitude of the tip residual load. The method also implicitly assumes a uniform distribution of shaft friction, by computing an average value for k_τ. The correlation presented for k_τ is quite misleading. The scatter shown on Figure (61) is large, so that the proposed correlation is not very representative for the measured values of k_τ that were obtained from pile load tests. It has also been shown that many uncertainties are associated with the number of blows obtained from the standard penetration test (e.g. by Schmertmann, 1975 and many others). Furthermore, it is not justified to use a strength indicator, like the SPT number of blows, to obtain a deformation parameter like k_τ. It is well known that the deformation characteristics of sand are very much affected by such factors as the fabric and the stress history,

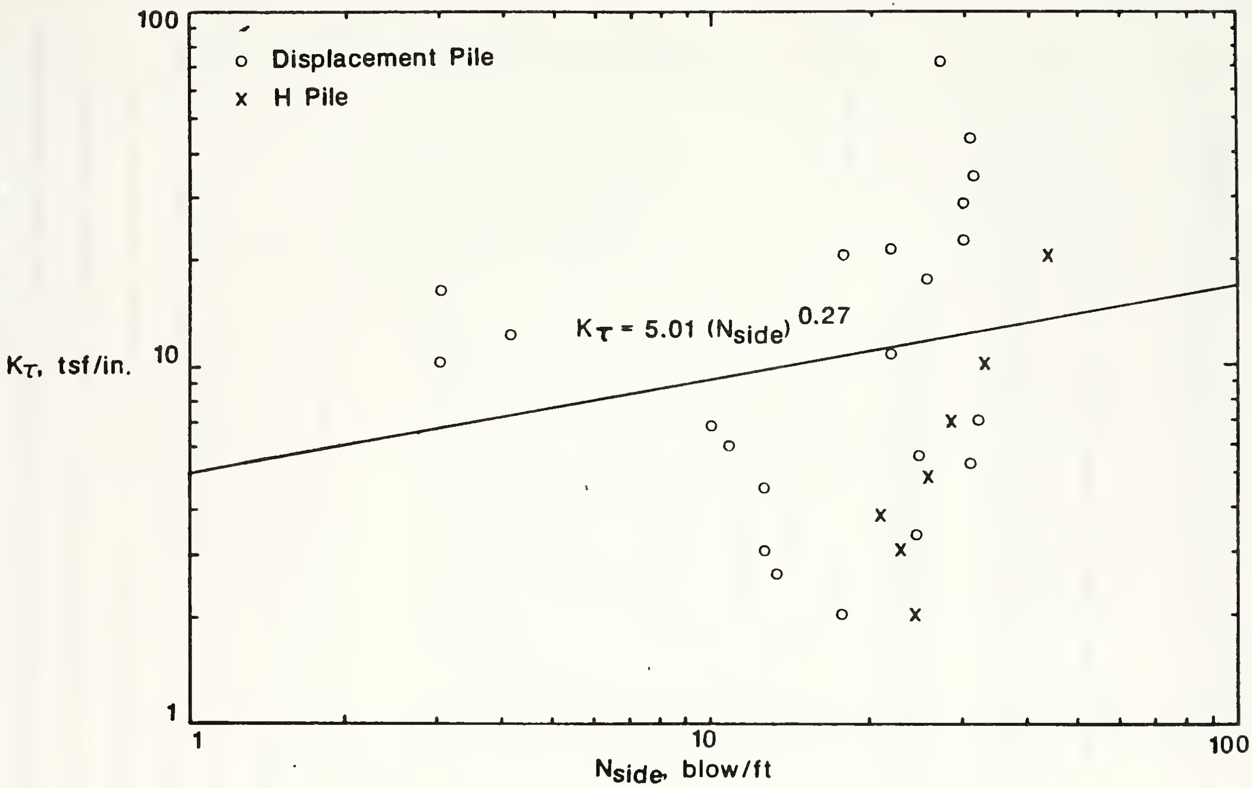


Fig. 61. Correlation Between N_{side} and K_τ (from Briaud and Tucker, 1984-b)

which may not be reflected on the SPT blow count (Leonards et al., 1986).

All the above mentioned considerations indicate that the Briaud-Tucker procedure for predicting residual stresses is very approximate and may sometimes give misleading predictions.

4.5.4 <u>The Procedure Suggested in this Research:</u>

The review of the existing methods for residual stresses predictions indicated the need for a more accurate procedure that gives a good approximation which can be used frequently without the use of a sophisticated computer analysis. It was also indicated that the wave equation analysis is the only approach that can be used for such prediction, taking into account all the key parameters that govern the magnitude of the residual forces. Therefore, this approach was used to develop a set of charts that can be easily used to make a reasonable prediction of the residual stresses.

Two pieces of information are required for such prediction, viz. the magnitude of the residual tip load, and the magnitude and distribution of the residual shaft loads. The latter can be easily determined if the residual point load is known. As was shown earlier, the shape of this distribution is unique, and similar to the one shown in Figure (52-a). Assuming that the residual point load "Q_{PR}" is known from the residual stress percent "a_r", that will be predicted later, the maximum residual shaft load "Q_{RMAX}" can be obtained from

$$Q_{RMAX} = b_r * Q_{PR} \tag{4.10}$$

where "b_r" is a factor which equals 1.05 for cohesionless soils, as concluded earlier.

The value of "Q_{RMAX}" occurs at a depth "Zcr" which can be determined from

$$Z_{cr} = b_c * D \tag{4.11}$$

where D is the pile length and b_c is a factor that can be obtained using Fig. (53-a) and Eq. (4.5) in terms of the pile length and the shaft friction percent "m". If the residual stress percent "a_r" were obtained from direct field measurements, Fig. (53-c) could be used together with Eq. (4.5). The location of the point of inflection (point "1" on Fig. (52)) at depth Z_m can be determined from

$$Z_m = b_m * Z_{cr} \tag{4-12}$$

where b_m = 0.60 for cohesionless soils.

Once all the above information is known, and considering the fact that the pile should be in equilibrium under these residual stresses (residual tip load plus the upward residual friction load should be equal to the downward friction load in the upper portion of the pile along the length Z_{cr}), the distribution of the shaft residual loads can be defined with fair accuracy.

In order to predict the residual stress percent "a_r", more than 250 computer runs were performed using the program "CUWEAP" described earlier. The parameters that were changed in this study were the pile material, the total pile capacity, the pile cross section, the pile length and the skin friction percent "m". It was shown that these are the controlling factors that affect the magnitude of the residual tip load. The abscissa to be used for the resulting relations was chosen to be the total pile capacity, since the shapes of the resulting curves are somewhat unique, with an extensive straight line portion, as was shown earlier. Hence, many curves were drawn between the total pile capacity and the residual stress percent for two pile materials, concrete with E = 5,000 ksi and steel with E = 30,000 ksi, for different pile lengths and cross sections. The skin friction percent was fixed at a value of 40%, considering to take its effect into account later. The resulting charts are shown on Fig. (62) and (63), corresponding to concrete and steel piles, respectively. For concrete piles having a value of E = 3,500 ksi, the values of "a_r" given in Fig. (62) should be increased by 10%, as explained earlier. For other values of E, linear interpolation could be used.

The effect of changing the value of the skin friction percent "m" can be taken into account using two factors β_m and β_m'

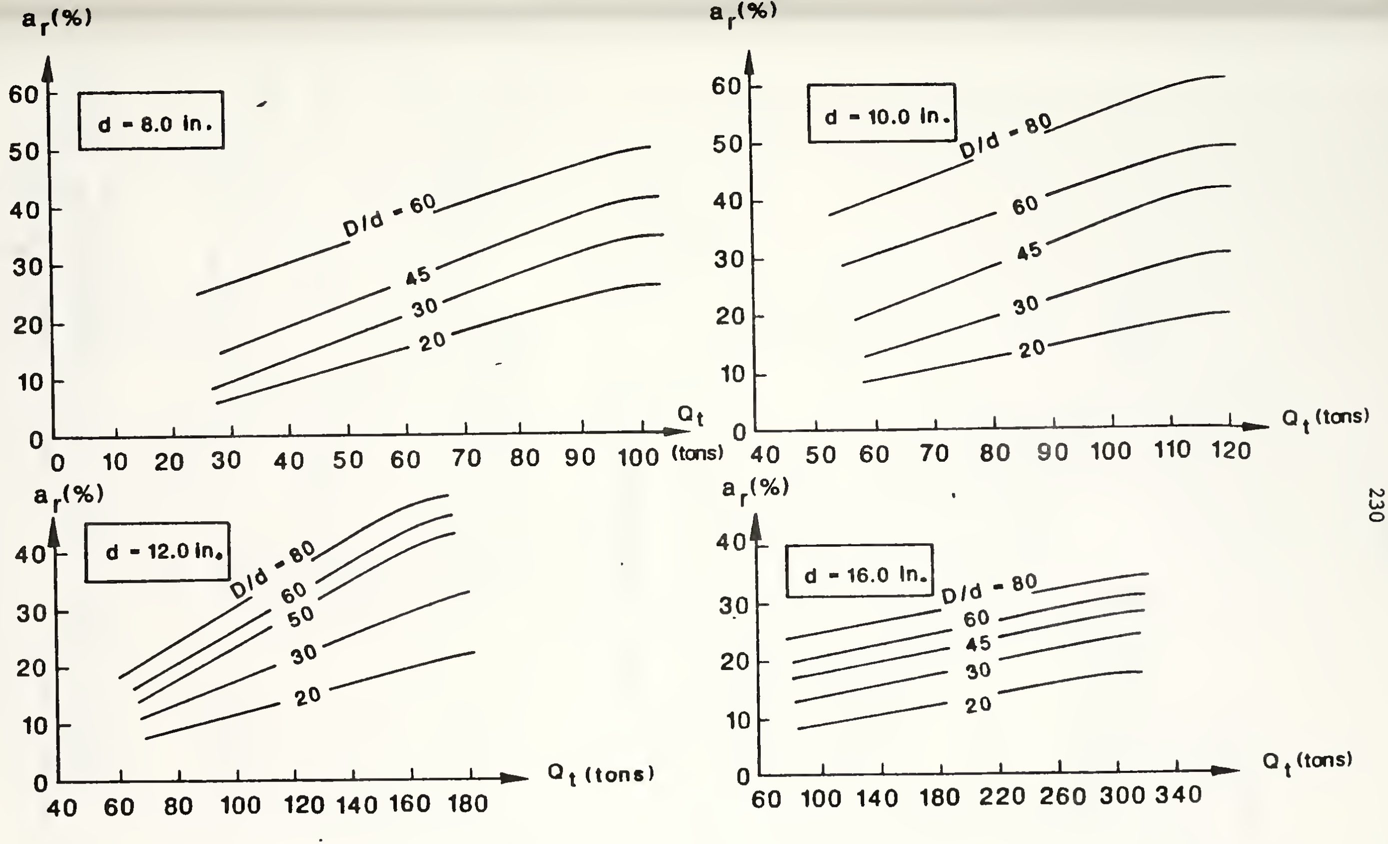


Fig. 62. Prediction Charts for the Residual Stress Percent "a_r" for Concrete Piles

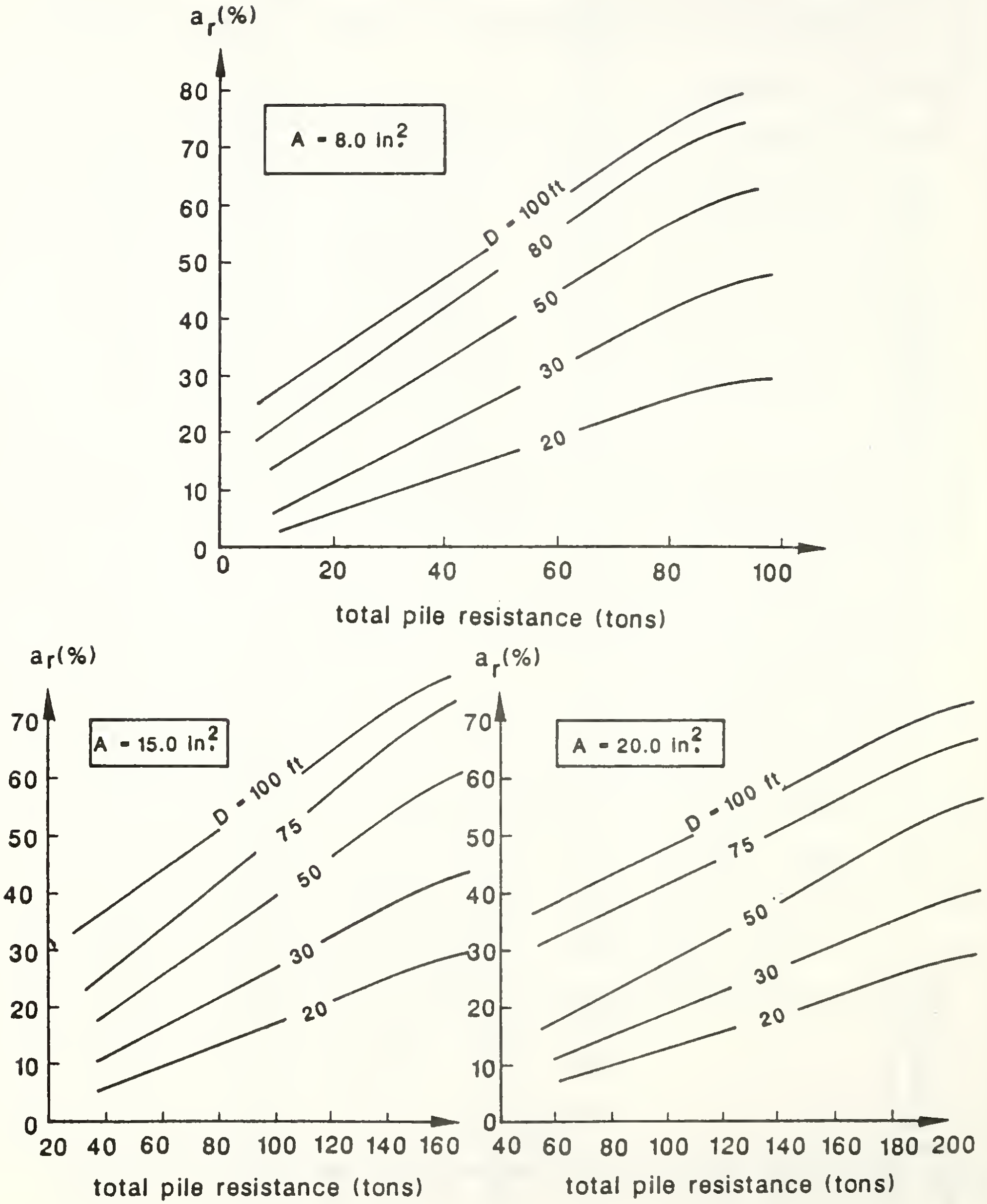


Fig. 63. Prediction Charts for the Residual Stress Percent "a_r"

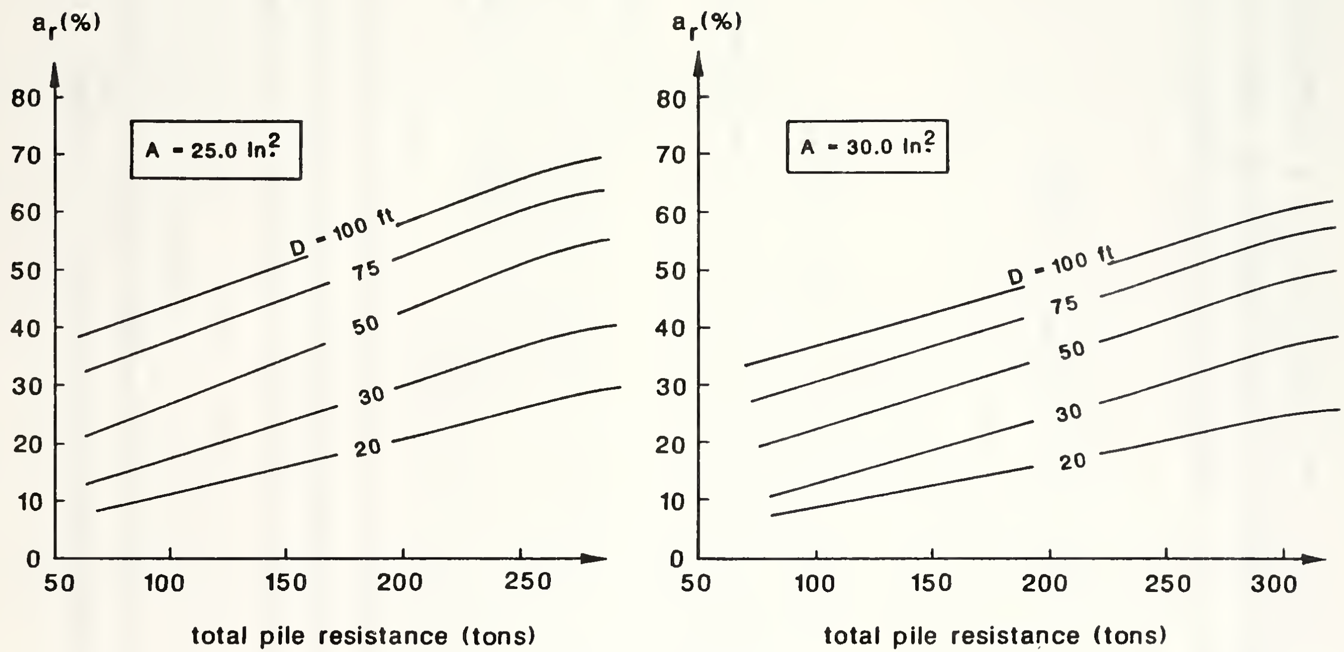


Fig. 63 (cont.). Prediction Charts for the Residual Stress Percent "a_r" for Steel Piles given in Fig. (64). If the value of "a_r" at m = 40% is obtained from the charts given in Fig. (62) and (63), the value of "a_r" at any other value of "m" larger than 20% can be obtained from the relation:

$$(a_r)_m = \beta_m * (a_r)_{40\%} \quad (m>20\%) \qquad (4-13)$$

If "m" was less than 20%, the actual value of "a_r" can be obtained from the relation:

$$(a_r)_m = 0.6 * \beta'_m * (a_r)_{40\%} \quad (m<20\%) \qquad (4-14)$$

Two situations can be expected during the design stage of the pile foundations:

1. The total pile capacity (Q_t), the load carried by the pile point (Q_p) and the load carried by the pile shaft (Q_s) are estimated by a reliable procedure, which does not take residual stresses into account.

2. Load test data on an instrumented pile were available, provided that the instrumentation was zeroed after the pile driving so that the residual stresses could not be measured.

The determination of the residual point load Q_{PR} is somewhat different for these two situations:

Situation No. 1:

The charts given in Fig. (62) and (63) are entered with the available pile data to get the value of "a_r" at m = 40%. Then Fig. (64) and Eq. (4-13) or (4-14) are used to obtain a_r at the expected value of m. The actual value of E should also be accounted for as explained earlier. Thus, Q_{PR} can be calculated from:

$$Q_{PR} = \frac{a_r}{100} * Q_p \tag{4-15}$$

After Q_{PR} is obtained, the distribution of the residual load along the entire pile shaft can be determined as explained earlier.

Situation No. 2:

If a load test has already been performed on an instrumented pile, and it is required to obtain the residual forces for a better interpretation of the results, a different procedure should be followed. This is because the skin friction percent that is obtained from the load test results represents the measured value of the shaft friction force which was affected by the existence of the residual stresses. The one used for developing the prediction charts, however, assumes the existence of no residual stresses. This means that the value of "m" is affected by the residual stresses as well as affecting them. Therefore, an iterative process is essential to get a good estimate about both "m" and a_r.

It is known that the measured value of point load Q_{p_m} is actually given by:

$$Q_{p_m} = Q_p - Q_{PR} \qquad (4\text{-}16)$$

where Q_p is the true point load with no residual stresses and Q_{PR} is the point residual load. The value of Q_{PR} can be expressed as a fraction of Q_p in terms of "a_r", and hence:

$$Q_{p_m} = Q_p - \frac{a_r}{100} * Q_p = (1 - \frac{a_r}{100})\ Q_p \qquad (4\text{-}17)$$

$$Q_{PR} = [\frac{a_r}{100} / (1 - \frac{a_r}{100})]\ \ Q_{p_m} \qquad (4\text{-}18)$$

or

$$Q_{PR} = A_r * Q_{p_m} \qquad (4\text{-}19)$$

where

$$A_r = \frac{a_r}{100-a_r} \quad \text{or} \quad a_r = \frac{A_r}{1+A_r} \qquad (4\text{-}20)$$

Therefore, the procedure for getting Q_{PR} should be as follows:

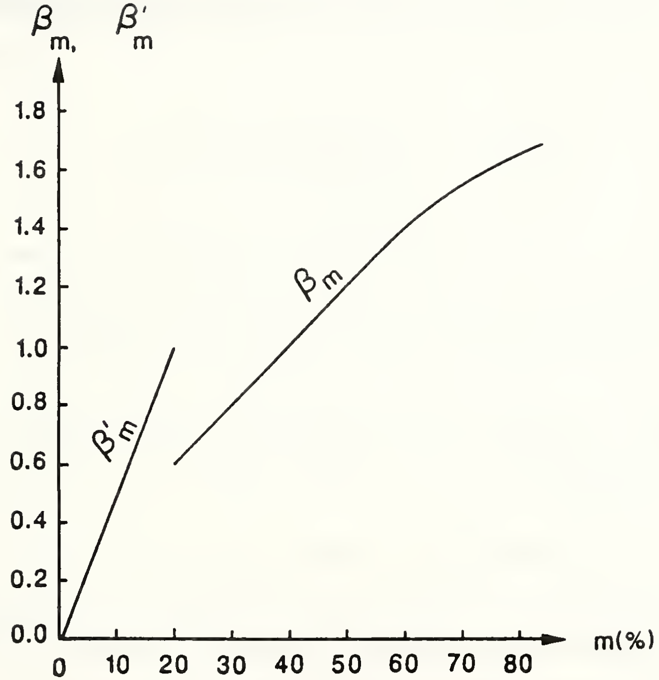


Fig. 64. Factors β_m and β'_m

- Obtain Q_{p_m}, Q_{s_m}, Q_t (measured value is also the true value) from the load test. Get m_m (measured value of m) by using:

$$m_m = \frac{Q_{sm}}{Q_t} \times 100 \qquad (4\text{-}21)$$

- Enter the prediction charts to obtain an initial value for a_r.

- Compute A_r from Eq. (4-20) and Q_{PR} from Eq. (4-19).

- Compute the first approximation of the true value of Q_p using Eq. (4-16). The modified value of Q_s is the difference between Q_t and the approximation obtained for Q_p. Use the modified value of Q_s to produce a modified value for m.

- Repeat the previous steps several times and acquire Q_{PR} from each iteration.

- Draw a curve between any value of Q_{PR} and the ratio between the next value and the value on the abscissa. At least three points are required to define this curve. The intersection between this curve and the horizontal line passing through a ratio of 1.0 gives the actual value of Q_{PR}.

Illustrative Example:

Assume the data reported by Vesic (1970) for the pile number H-12. This pile has the following characteristics: steel pipe, d = 1.5 ft, D = 20.1 ft, A_p = 27.49 in^2 (modified for the effect of the instruments, after Briaud et al., 1983), E = 30,000 ksi. The values measured from the load test were: Q_t = 232 t, Q_{p_m} = 173 t, Q_s = 59 t, which means that m_m = 25.4%. The charts given in Figure (62) indicate that a_r for m = 40% equals 22%. Figure (64) indicates that β_m = 0.7, hence a_r = 0.7 x 22.0 = 15.4%. Therefore

$$A_r = \frac{15.4}{100-15.4} = 0.182$$

and

$$Q_{PR} = A_r * Q_{p_m} = 0.182 \times 173 = \underline{31.5t}$$

The first approximation for Q_p is (173 + 31.5) = 204.5t

Repeating the previous steps results in the following results:

Q_p = 204.5t → Q_s = 27.5t → m = 11.9% → a_r = 7.7%

A_r = 0.082 → Q_{PR} = <u>14.3t</u>

Q_p = 187.3t → Q_s = 44.7t → m = 19.3% → a_r = 12.9%

A_r = 0.148 → Q_{PR} = <u>25.7t</u>

$Q_p = 198.7t \rightarrow Q_s = 33.3t \rightarrow m = 14.4\% \rightarrow a_r = 9.5\%$

$A_r = 0.105 \rightarrow Q_{PR} = \underline{18.2t}$

The curve shown in Fig. (65) is drawn using the four underlined values obtained for Q_{PR}. The intersection point gives a value of $Q_{PR} = 21.7$ t. Hence the final results of the analysis are: $Q_p = 194.7$ t, $Q_s = 37.3$ t, m = 16.1% and $a_r = 11.1\%$. The results of this example demonstrate the effect of the residual stresses on the interpretation of the load test results.

It should be noted that if the predicted value of a_r is somewhat high, one may face some numerical difficulties when using the above outlined procedure. This is because the value of A_r in this case would be very large, and hence the value of Q_{PR} obtained within a certain trial would be unrealistically high. One should remember, however, that the residual tip load cannot exceed the shaft friction load (in fact it is usually smaller). Hence, to reduce the number of iterations, a reasonable lower value of "m" can be assumed to reduce the value of a_r obtained, and then proceed with the iterations as usual.

Examination of the proposed procedure for the residual stresses prediction indicates many advantages. First, it is simple to apply and can be used in a matter of minutes. Second, it includes the effect of all the factors that affects the residual stresses, especially the distribution of pile capacity between

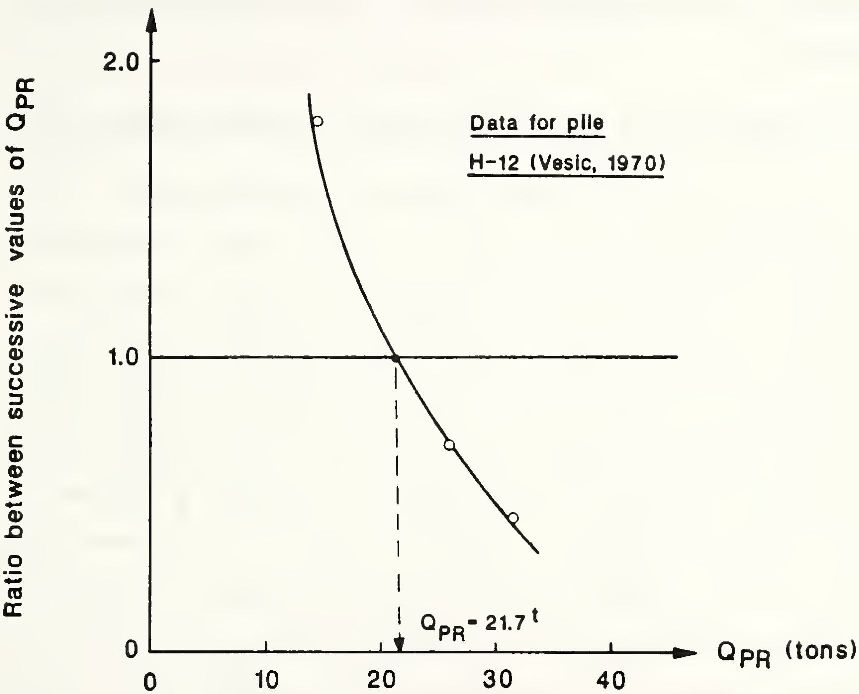


Fig. 65. Iterative Procedure for Residual Point Load Prediction from Load Test Results

point load and side friction. And finally, it does not depend on results of any in situ tests that may have many uncertainties, like the standard penetration test. The final step is to compare this method with available field test data and with the other procedures that have been proposed for the residual stresses prediction.

4.5.5 Comparison of Proposed Procedure with Other Techniques:

The different procedures that have been suggested for the residual stresses determination, including direct measurements, were used to compute these stresses for different piles with available load test data. Table (19) shows the data of the instrumented test piles that could be located in the literature such that residual stresses could be predicted by one or more of the available prediction techniques. These data were used to predict the tip residual forces for the piles using several procedures. The methods used were: the direct measurements of the tip residual load by means of load cells zeroed before driving the pile, the Hunter-Davisson procedure, the Holloway procedure (two predictions were made using this procedure, one by using actual static test measurements and the other by using the static load test results predictions made by Holloway et al., 1975), the Briaud-Tucker procedure and the procedure recommended during the course of this study. Table (20) summarizes the values of the residual stress percent "a_r" obtained by the different procedures. It can be noticed that the number of values obtained by

Table (19) Pile Data Used for Residual Stress Analysis

Site and Reference (1)	Pile No. (2)	Pile Type and Material (3)	Pile Characteristics: Diameter* (ft) (4)	Pile Characteristics: Length (ft) (5)	Pile Characteristics: x.sec.** area (in^2) (6)	Pile Characteristics: Modulus of elasticity ($psi \times 10^6$) (7)	Average "N" from SPT (blows/ft) (8)	Total Pile Capacity, Load Test (tons) (9)	Measured Skin Friction Percent (m%) (10)
Low Sill Structure Old River (Furlow, 1968)	5	Steel pipe	1.42	45.1	22.65	29.0	na+	144	63.2
Lock and Dam 4, Arkansas River (Hunter & Davisson, 1969)	1	Steel pipe	1.20	53.1	17.12	29.0	27	172	72.0
	2	Steel pipe	1.50	52.8	23.86	29.0	27	251	72.0
	3	Steel pipe	1.70	53.0	27.36	29.0	27	258	56.6
	7	Steel "H", HP 14x73	1.34	52.1	29.33	29.0	27	220	70.5
	16	Steel pipe	1.42	52.7	19.62	29.0	27	165	69.7
Ogeechee River (Vesic, 1970)	H-11	Steel pipe	1.50	9.9	27.49	30.0	6.7	76	19.7
	H-12	Steel pipe	1.50	20.1	27.49	30.0	12	232	25.4
	H-13	Steel pipe	1.50	29.1	27.49	30.0	13	297	28.6
	H-14	Steel pipe	1.50	39.3	27.49	30.0	18.7	347	38.3
	H-15	Steel pipe	1.50	49.3	27.49	30.0	23.5	421	38.7
St. Charles River, Quebec (Tavenas, 1971)	H5	Steel "H" 12BP74	1.09	60.0	21.8	35.0	25	78	52.0
	J5	Hexagonal, concrete	1.05	60.0	127.0	3.94	25	150	58.0

* Equivalent diameter for piles other than circular
** Adjusted for instrumentation effect (Briaud et al. 1983)
\+ This part of data was not available to the author

Table (19) Continued

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Drammen River, Norway (Gregersen et al., 1973	A	Circular, concrete	0.92	26.2	95.45	3.15	2.3	30	80.0
	D/A	Circular, concrete	0.92	52.5	94.45	3.15	3.65	50	77.2
Sweden (Sellgren, 1982)	A-I	Square, concrete	1.00	35.4	113.0	3.15	na	77.5	14.8
	A-II	Square, concrete	1.00	35.4	113.0	3.15	na	142.5	76.5
West Seattle (Gurtowski and Wu, 1984)	A	Octagonal, concrete	2.05	98.0	477.2	5.56	35	520	85.0
	B	Octagonal, concrete	2.05	84.0	477.2	5.56	38	450	83.3
Lower Arrow Lake B.C., Canada (McCammon and Golder, 1970	1	Steel pipe	2.0	150.0	37.7	30.0	12	504	36.5

Table (20) The Residual Stress Percent "a_r" by Different Methods

Site and Reference	Pile No.	Residual Stress Percent					
		Direct Measurement	Hunter and Davisson	Holloway, with Measured Test Data	Holloway, with Predicted Test Curve	Briaud and Tucker	Suggested Method
Low Sill Structure Old River (Furlow, 1968)	5	-	53.7	-	-	-	53.9
Lock and Dam 4 Arkansas River (Hunter and Davisson, 1969)	1	-	43.5	50.3	72.8	24.7	56.8
	2	-	37.5	50.0	73.6	23.6	60.5
	3	-	30.0	38.0	36.6	21.0	47.7
	7	-	27.8	-	-	19.3	50.8
	16	-	44.4	57.6	74.0	30.75	49.0
Ogeechee River (Vesic, 1970)	H-11	-	-	-	-	5.1	2.9
	H-12	-	-	-	-	4.0	11.1
	H-13	-	-	-	-	5.0	16.7
	H-14	-	-	-	-	6.5	24.2
	H-15	-	-	-	-	7.0	30.0
St. Charles River Quebec (Tavenas, 1971)	H5	-	-	-	-	13.4	30.8
	J5	-	-	-	-	15.1	38.8
Drammen River, Norway (Gregersen et al., 1973)	A	29.0	-	-	-	35.2	10.70
	D/A	31.7	-	-	-	32.7	35.5
Sweden (Sellgren, 1982)	A-I	10.6	-	-	-	-	7.0
	A-II	38.8	-	-	-	-	46.2
West Seattle (Gurtowski and Wu, 1984)	A	40.0	-	-	-	30.5	47.0
	B	40.0	-	-	-	27.8	43.0
Lower Arrow Lake B.C., Canada (McCammon and Golder, 1970)	1	-	-	-	-	22.8	26.4

direct measurements is quite limited, which makes it difficult to evaluate the various prediction techniques. It is of extreme importance to perform load tests on driven piles in which the instrumentation is zeroed before driving, at least for research purposes. Reliable instruments should be developed such that they are not affected by the driving process and hence can give reliable measurements for residual stresses.

Figure (66) shows the relationship between the predictions made by the procedure recommended in this study and actual measurements taken for the residual stress percent, either measured directly or obtained indirectly by the Hunter-Davisson method. The comparison with the direct measurements (given by solid points) indicates a very good agreement, except for one data point reported by Gregersen et al., (1973) for pile number A. A careful look at the data, however, shows that the measurements of a_r for piles (A) and (D/A) are quite similar, although the second pile is twice as long and was driven to about 67% higher resistance than the first one. This discrepancy suggests that the measurements for pile "A" may not be correct, especially since the measurements for pile (D/A) were in good agreement with the prediction. On the other hand, some differences can be observed between the predictions and the indirect measurements obtained by the Hunter-Davisson procedure. Two piles showed good agreement, while four piles indicated predictions which are generally higher than the indirect measurements. As mentioned earlier, there are

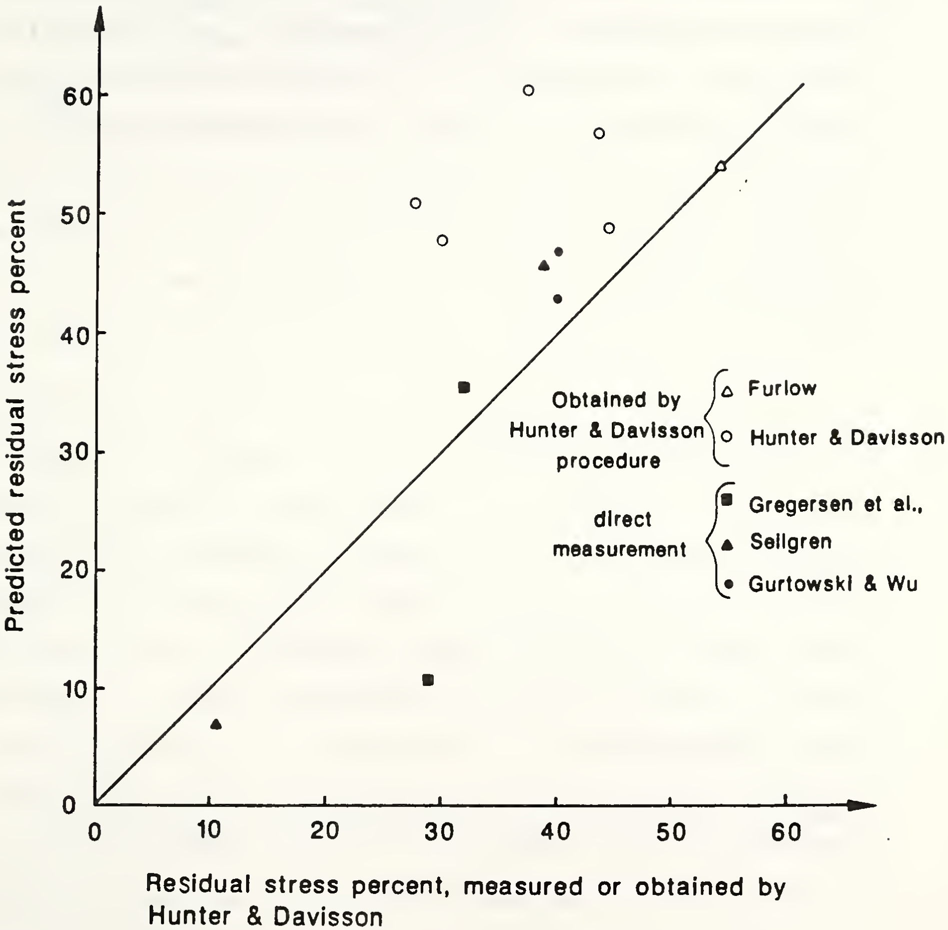


Fig. 66. Predicted vs Measured Residual Stress Percent

several limitations for the Hunter-Davisson procedure. On the other hand, the data for which this procedure was used were based on measurements made during the 1960's while the direct measurements were made in the 1970's and 1980's. This suggests that the instruments used for the direct measurement were more reliable. Based on this discussion, one may conclude that the suggested prediction technique is in good agreement with the actual measurements, and can be used reliably for predicting the residual stresses. More tests, however, need to be performed to gather more data of either direct or indirect measurements of the tip residual stresses so that more comparisons could be made.

Figure (67) shows the comparison between all available techniques for obtaining residual stresses, relative to the method suggested in this study. The first observation that can be drawn from this figure is that the suggested prediction gives the closest values of "a_r" to the direct measurements. Next come the values predicted by the Holloway procedure, if the actual static load test results are used. As mentioned earlier, the basic idea of the predictions made by Holloway et al., (1975) and by the suggested method is the same. However, the wave equation model used for this study is superior to the one used by Holloway. On the other hand, if the static load test results were predicted by the computer program "DUKFOR", rather than being directly measured, the prediction based on the Holloway procedure was not very good. This is attributed to the large approximations used in the

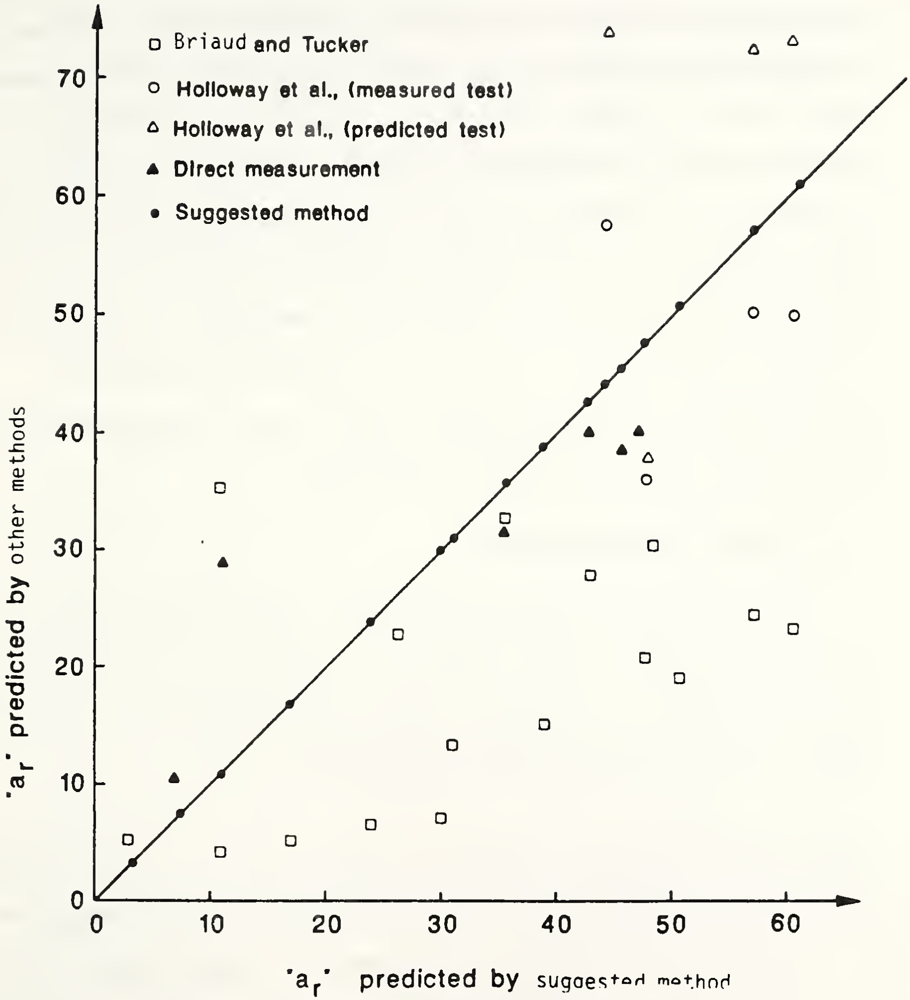


Fig. 67. Comparison between Different Methods for Residual Stress Prediction

analyses performed by Holloway et al., (1975,1978) regarding the soil-pile model (using the transfer function approach). Finally, the values obtained by the Briaud-Tucker technique were quite far from the other predictions, as well as from the direct measurements. They generally tend to be lower (i.e. this method underestimates the residual stresses). Several discrepancies can be observed in Table (20), e.g. the predictions for the Ogeechee River piles were not very sensitive to the variation of the pile length and the soil resistance as the recommended procedure was. As mentioned earlier, there are so many disadvantages and shortcomings for this procedure that it may be misleading to use it for predicting the residual stresses.

4.6 Summary and Conclusions

The residual forces were obtained below the pile tip and along its shaft for piles driven into cohesionless soils using the wave equation approach. The following results and conclusions were obtained from this analysis:

1. The existence of residual stresses due to the pile driving process affects both the static behavior and the driveability of the pile. Although the total pile capacity should be the same, the distribution of this capacity along the pile shaft and below the tip may change considerably due to the existence of the residual stresses. This change has an effect on many aspects of the pile foundation design (e.g.

settlement, negative skin friction, etc.). On the other hand, the existence of residual stresses facilitates the driving procedure. In other words, higher resistance can be reached with a smaller number of blows. In the meantime, the driving stresses may increase by about 5% on the average (up to as much as about 15%) due to the existence of the residual stresses. Hence, it is of extreme importance to include these stresses in both static and dynamic analyses of the pile.

2. The general shape of the residual force distribution along the pile shaft does not vary, irrespective of the variables involved in the problem. Only the magnitude of these forces changes, according to the variations in the different parameters. This distribution indicates the existence of a negative friction for the upper portion of the pile, down to a critical depth "Z_{cr}", below which there is as upward residual friction load, in addition to the residual tip load. This study suggested values for shape factors, b_r, b_c, and b_m, by which the shape and the magnitudes of the residual forces along the pile shaft for piles in cohesionless soils can be approximately determined, provided that the residual tip load is known.

3. The study showed that the main factors that affect the magnitude and distribution of residual stresses are the total soil resistance, the percentage of load transferred by

shaft friction, and the pile material, length and cross section. The effect of the driving system and driving components is minor. It was found that residual stresses increase as the total soil resistance increases, as the percentage of load carried by skin friction increases, as the pile length increases and as the cross sectional area decreases. The pile material greatly affects the residual forces. As the pile modulus of elasticity increases, the residual stresses decreases holding other parameters constant.

4. This study led to the development of easy-to-use charts for the residual stresses prediction. It was felt necessary to develop such a procedure for many reasons. First, it is not a common practice to perform pile load tests at the design stage on instrumented piles, for which the instrumentation is zeroed before driving. This means that residual stresses cannot be obtained. Approximate procedures used for that purpose, e.g. the Hunter-Davisson procedure, make simplifying assumptions that can lead to incorrect determination of the residual stresses. The difficulty of obtaining residual stresses from pile load tests could be overcome by using the wave equation analysis for their predictions. Computer programs like "DUKFOR" and "CUWEAP" have proven to be successful in this respect. However, they require the availability of both the software and the

hardware necessary for their operation, which costs both time and money, limiting their use in the everyday practice. A prediction technique suggested by Briaud and Tucker (1984-b) was based on the average value of the number of blows/ft obtained from the Standard Penetration Test. Some important parameters, e.g. the skin friction percentage and the total pile capacity, were not taken into account in this procedure. A large scatter was observed for the data used to develop these correlations. Furthermore, the Standard Penetration Test results cannot be justified for use in predicting deformation characteristics of the soil, in addition to all the uncertainties associated with this test. For these reasons, the method suggested in this study, which is based on an improved wave equation analysis, provides a simple approach for predicting residual stresses for driven piles.

5. Comparisons of actual field measurements with the various prediction techniques showed that the author's suggested method gave predictions which are the closest to the measurements. These actual measurements, however, were very limited in number. It is recommended that more load tests on instrumented piles, for which the instrumentation is zeroed before driving, be conducted. The results of these tests can be used for more comparisons with the suggested prediction. It is of crucial importance to develop reli-

able instruments for this purpose, so that they will not be severely affected by the driving process.

CHAPTER 5

SUMMARY AND CONCLUSIONS

This report was prepared to complete the study "Computational Package for Predicting Pile Stress and Capacity." While the interim report (Tejidor, 1984) described the analytical determination of pile capacity by means of the computer program "PPILE," this report deals mainly with both static and dynamic load testing. These tests provide the best evidence of pile capacity and serve as a reference of accuracy for other load prediction techniques.

5.1 Summary:

The main objectives of this study were:

1. To emphasize the importance of performing pile load tests for obtaining the most valuable and accurate data for the design of pile foundations.

2. To present the state-of-the-art of pile load testing, regarding the equipment used, the instrumentation, the testing procedures and the interpretation techniques.

3. To recommend quick, inexpensive methods for pile testing by IDOH for the sake of minimizing the costs without compromising the data obtained.

4. To familiarize the IDOH with the state-of-the-art of performing dynamic measurements during pile driving, to briefly illustrate the theoretical background behind their use and to show the potential uses of these measurements.

5. To study the subject of residual stresses due to driving, since they greatly affect the interpretation of static load tests, as well as the mechanism of load transfer, and to develop a simple approach for the prediction of such measurements.

6. To develop a simplified procedure to account for the existence of negative skin friction and to compute the resulting additional loads.

5.1.1 Static Pile Load Tests:

The importance of routine performance of pile load tests, even for small scale jobs, was emphasized in the report. Planning the testing program and the application of test results were discussed. Emphasis was given to axial compression load tests, although other forms of tests, e.g. lateral, uplift and torsional testing, were described. For each type of test, the state-of-the-art information about the following items was given: the loading systems; the measurement of pile movements; the potential sources of error; and the available testing procedures with the interpretation of their results. For axial load tests, the following methods were described: the maintained loading tests

(ML);; the constant rate of penetration test (CRP); the method of equilibrium; and the Texas Highway Department quick testing method. It was shown that the use of quick load testing techniques correlates well with the traditional time-consuming methods, besides having the advantage of being much cheaper, which justifies their routine use for all types of jobs involving pile foundations. Based on these studies, recommendations were made to the IDOH regarding the procedure of performing pile load tests.

5.1.2 Dynamic Measurements for Pile Driving:

Another type of pile testing involves the use of dynamic measurements during pile driving. These techniques are the best ones introduced thus far for monitoring the pile during driving. They can be applied, together with the wave equation analysis, in a variety of ways. Among their uses are pile capacity predictions; the evaluation of the driving system with respect to the hammer efficiency and performance, cushions, capblocks, etc; measurement of pile stresses; and the verification of the pile integrity.

The historical background of equipment development was described. The most recent advances for force, velocity and acceleration measurements, together with the appropriate recording devices were reviewed. A brief description of the theoretical background behind these measurements was given. Since the purpose of discussing dynamic measurements in this report was to familiarize the IDOH with the subject in an easy-to-follow manner, some of the complicated mathematical derivations and expressions were omitted to help simplify the subject.

Dynamic measurements have been used to predict the geotechnical pile capacity, either in situ using a field computer and the approximate CASE method, or in the office using a more sophisticated analysis (CAPWAP). The latter analysis can also be used to predict the load transfer along the pile shaft and the load deformation curve that would be obtained from a pile load tests. Dynamic measurements have been also used to monitor driving hammers; evaluate their efficiencies under different operating pressures, strokes or batters; and to check the driving elements, i.e., cushions, capblocks, etc. Finally, dynamic measurements have been used to examine the performance of the pile, detect any damage and evaluate the actual pile lengths if not known.

5.1.3 Residual Stresses Due to Pile Driving:

To obtain a better interpretation of the static load test results, the report also discussed the subject of residual stresses due to driving. The main factors affecting these stresses were described. These factors were found to be the total soil resistance, the percent skin friction, the pile length, the pile cross sectional area and the pile material. The methods that have been suggested thus far for the residual stresses prediction were reviewed. The review showed that none of the available methods can give satisfactory predictions. Based on extensive parametric studies, a new procedure was developed at Purdue for the prediction of magnitude and distribution of residual stresses. This procedure was introduced by means of easy-to-use charts and nomograms, with the help of some illustrative examples. Predictions made by this technique were compared with actual measurements and good agreements were proven.

5.1.4 Negative Skin Friction:

Finally, a computer program PPILENF for the prediction of additional pile loads due to negative skin friction was developed. Because there are many uncertainties regarding the available methods of predicting negative skin friction loads, upper bound values were used to develop this program. Complete listing of the program is given in the Appendix A together with User's manual, input forms, and illustrative examples.

5.2 Conclusions:

5.2.1 Pile Load Tests:

1. Pile load tests are very useful, and sometimes essential, in all stages of design and construction of pile foundations. They should be routinely used to get better predictions of the pile capacity and short-term settlement, and

hence to permit better design and avoid unnecessary costs due to conservative predictions.

2. In order to minimize the costs involved in performing pile load tests, quick load testing procedures have been developed. Examples of these include the constant rate of penetration test (CRP), the Texas quick load tests, and the method of equilibrium.

3. It is recommended that the IDOH use either the quick load test method or the method of equilibrium, depending on the pile-soil system and the purpose of conducting the test. In general, the quick tests method can be used for proof testing and in the case where no settlement data are required. If allowable settlement is the main design criterion, the method of equilibrium should be used. Specifications for both methods were described in detail within the report, together with illustrative examples.

5.2.2 Dynamic Measurements:

1. Dynamic measurements constitute a very efficient way of monitoring the pile during driving.

2. The main quantities to be measured are the forces, displacements, velocities and/or accelerations. These measurements are usually taken at the pile top.

3. The main uses of dynamic measurements are the prediction of pile capacity and load transfer, the evaluation of the efficiency of the driving hammer and elements, and the examination of the integrity of the pile to detect any possible damage.

4. The prediction of pile capacity by this technique is much better than the use of a driving formula (e.g. ENR, Hiley, etc.). On the other hand, it is not accurate enough to be used alone without other techniques of pile design. This is primarily due to the insufficiency of the soil model used. The prediction by this technique, however, completes the spectrum of pile design procedures and gives more confidence in the other procedures, since it is based on direct measurements during the driving process. On the other hand, dynamic measurements have proven to be quite successful in determining the wave equation parameters, measuring the actual energy delivered to the pile, monitoring the driving process with all of its elements, measuring the pile stresses and performing quality control procedures.

5. It is recommended that the IDOH acquire the equipment used for dynamic measurements and prepare the required personnel with the appropriate training. The savings that can be achieved by using these measurements in several jobs would very soon cover the price of the equipment. More

importantly, the improvement in design and execution procedures of the pile foundations which will not only save money in the short term, but will also reduce the maintenance and replacement costs that might have been otherwise necessary in the long run.

5.2.3 Residual Stresses Due to Pile Driving:

1. At the start of loading, a driven pile, either in a static load test or by the load coming from the superstructure, is not stress-free as often assumed. Residual point and shaft stresses accumulate during driving. Although the total capacity of the pile is not changed, these stresses can result in a substantial change in the mechanism of load transfer between the pile and the soil, and consequently the settlement of the pile group. The observed tip load is lower and the observed shaft friction is higher than the true values.

2. The existence of residual stresses affects the driveability of the pile. Higher resistance can be reached with a smaller number of blows. In the meantime, the driving stresses may increase by about 5% on the average, (to as much as about 15%) due to the existence of residual stresses. Hence, it is of extreme importance to include these stresses in both static and dynamic analyses of the pile.

3. The general shape of the residual force distribution along the pile shaft does not vary, irrespective of the variables involved in the problem. Only the magnitude of these stresses change, according to the variations in the different parameters.

4. The magnitude of the residual loads increases: as the total soil resistance increases, as the length of the pile increases, as the cross sectional area of the pile decreases, as the elastic modulus of the pile decreases, and as the percentage of load transferred by the shaft friction increases. The effect of the driving system and driving components is minor.

5. The available techniques for predicting residual stresses are not satisfactory. Hence, a prediction technique based on easy-to-use charts and nomograms was developed at Purdue. Comparisons of actual field measurements with the various prediction techniques showed that the suggested method gave predictions which are the closest to the measurements.

5.2.4 Negative Skin Friction:

1. Additional loads due to negative skin friction can lead to foundation failures if not taken into account in design.

2. Available techniques for predicting these loads are not satisfactory and additional research is needed to overcome this problem. Until this research is developed, it is recommended that an upper-bound prediction be used for a conservative design.

3. A computer program PPILENF was developed at Purdue to give adequate prediction of additional loads due to negative skin friction. It is recommended that the IDOH use this program for the design of pile foundations in the situations where negative friction is involved. This will avoid serious prediction errors and long term problems.

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APPENDIX A-1

Pile Loads Due to Negative Skin Friction

Negative skin friction, if not considered in the design of the pile foundation, may result in excessive deformations or even failure of the foundation. A summary of the phenomenon and a review of some of the methods that have been used to calculate the magnitude of the negative friction is presented in Tejidor (1984).

The calculation of the amount of additional negative friction force transferred to the pile group is still beyond the state-of-the-art. The magnitude and distribution of the negative skin friction stress is a result of a complex combination of factors that are not yet will understood. When the designer is faced with a situation where it is necessary to consider the negative skin friction, two main problems arise:

1. How to identify the zone within which the negative friction will act. This is dependent on the soil layers involved, the relative displacement between the pile and the soil, and the pile length. The negative friction develops only along the portion of the pile shaft where the soil settlement exceeds the downward displacement of the pile shaft. Therefore, below a certain "neutral point" there would not be enough relative movement between the pile and

the surrounding soil and the friction remains positive. A wide variety of positions was suggested for the location of this neutral point (Buisson et al., 1970; Gognan, 1972; Bozozuk, 1970; Garlanger, 1973; etc.). The uncertainty is enhanced when the soil nature is relatively heterogeneous. It is influenced by factors such as relative compressibility of the pile shaft and underlying soil with respect to the surrounding soil, relative magnitude of axial load in the pile with respect to the effective stress change that causes settlement of surrounding soil, as well as the position of the most compressible stratum in the overall soil profile (Bozozuk, 1970; Buisson et al., 1960).

2. The second problem is the calculation of the magnitude of the downward drag created by negative friction. Empirical methods have been suggested (e.g. Bowles, 1982) as well as theoretical methods based on the assumptions of the classical theory of elasticity (refer to Poulos and Davis, 1980). These methods, however, are complicated, and the assumptions involved may not correspond closely to reality. It should also be noticed that the magnitude of the negative friction is dependent on the horizontal stresses that develop in the long term (not just after driving). Some methods account for this effect but they are uncertain and not reliable (Leonards, 1985).

Due to the above mentioned problems, the only acceptable way to include the effect of negative friction is to use upperbound values. It should be noticed that the magnitude of negative friction is limited by the ultimate shaft friction that can be mobilized, and the magnitude of the forces causing the ground to subside (weight of dry fill, ground water lowering, etc.).

Leonards (1985) recommended the following procedure to get upperbound values of the amount of negative friction:

1. When the negative friction is caused by a dry fill layer, which is susceptible to large future amounts of settlement, the extra load that should be added to the original group load is the effective weight of the fill that is confined between two areas. The first one is the pile group area and the second is the area obtained by a downward slope of 45^{o} from the group area over the thickness of the fill layer.

2. For the case of groundwater lowering (where the effective weight of the soil increases) the additional load is the increase in the vertical effective stress times the area of the pile group.

3. For the case of soft or sensitive clay layer subject to remolding during pile driving or future settlement due to loads transmitted from the piles, the additional weight is

the effective unit weight of the soil layer times the area of the pile group times the height of the layer.

4. In some cases, a fill working platform is required to be placed over the surficial soft layer to facilitate the installation procedure. Negative friction may be expected in this case due to the fill and the soft layer. The upperbound load that should be added is the sum of the loads calculated in steps (1) and (3).

It should be noted that for the case of battered piles, the problem is so complex that it has not yet been dealt with in research (Leonards, 1985). No technique has been proposed to solve such a problem. Research is needed to provide even an approximate solution for this case.

A computer program PPILENF was developed at Purdue to calculate the additional loads due to negative skin friction. The need to produce another program for the negative friction, instead of simply modifying the original PPILE program arises for two reasons:

1. The static load obtained from static analysis is used later in the program to assess the driving conditions of the pile. Since the additional load due to negative friction is anticipated to take effect after the piles have been driven, it would be erroneous to incorporate the

distribution of shaft resistance determined from a negative friction evaluation into the wave equation analysis.

2. Since the amount of negative friction is highly dependent on the pile group characteristics (pile number and spacing, etc.), and since the PPILE analysis is only a single-pile analysis, it is not possible to evaluate the effect of negative friction correctly from a modified PPILE analysis. Hence, the designer would obtain the ultimate single pile capacity from PPILE, assume reasonable factors of safety and design the group configuration according to the appropriate considerations of settlement, economy, etc. Next he would use the PPILENF to evaluate the additional loads due to negative skin friction and modify the design accordingly.

The PPILENF program incorporates the methods of calculation suggested by Leonards (1985). Since the effect of the negative friction is not known in advance, and since the group design and the magnitude of the additional force due to negative friction are interrelated, an iterative procedure should be used. The number of piles is assumed according to the anticipated column load, with some allowance of an additional force due to negative friction. Next, the data are input into the computer, and one run of PPILENF is performed. The assumed number of piles would automatically be checked by the program and a printout message would appear to state if the assumed number of piles is reasonable, too high or too low. The output also would include the additional single pile and group load due to negative skin friction. If the assumed number of piles is inappropriate, the group design should be modified according to the information given by the output. The use of the program will be illustrated by examples.

It should be noticed that when the analysis of PPILE (for a single pile) is performed, no positive friction should be considered to be provided by the layers that are anticipated to produce negative skin friction when the static capacity is calculated using PPILE.

The program assumes that the allowable pile load can be increased by about 10% since the calculated additional loads are on the conservative side. Also the structural capacity of the pile under the additional negative friction loads should be checked, since this is not included in the program.

Users' Manual:

A complete listing of the PPILENF is submitted with the Appendix. The software was written in FORTRAN language. It was written specifically for the CDC 6500 computer system at Purdue. It can be used on the IDOH IBM 370 system.

An input form is also submitted. The variables required to be input on each card are listed as follows:

N The number of piles in the pile group

NF An integer variable that specifies the case for which the negative friction is calculated. It can have a value of 1, 2, 3 or 4 as follows;

´1´ - for the case of dry fill.

´2´ - for the case of groundwater lowering.

´3´ - for the case of soft or sensitive clay layer.

´4´ - for the case of fill overlying a soft layer.

H The thickness of the layer causing negative friction. It needs to be input only for NF=1,2,3 (ft).

A Area of the pile group (ft^2).

p Pile group perimeter (ft).

PAll Allowable pile load (ton).

GP Load required to be carried by the pile group (ton).

TITLE Title of the analysis (not more than 40 characters, including spacings).

GAMF Effective unit weight of fill, to be input for NF=1,4 (pcf).

GAMW Unit weight of water, to be input for NF=2 (pcf).

GAMM Effective unit weight of soft layer, to be input for NF=4 only (ft).

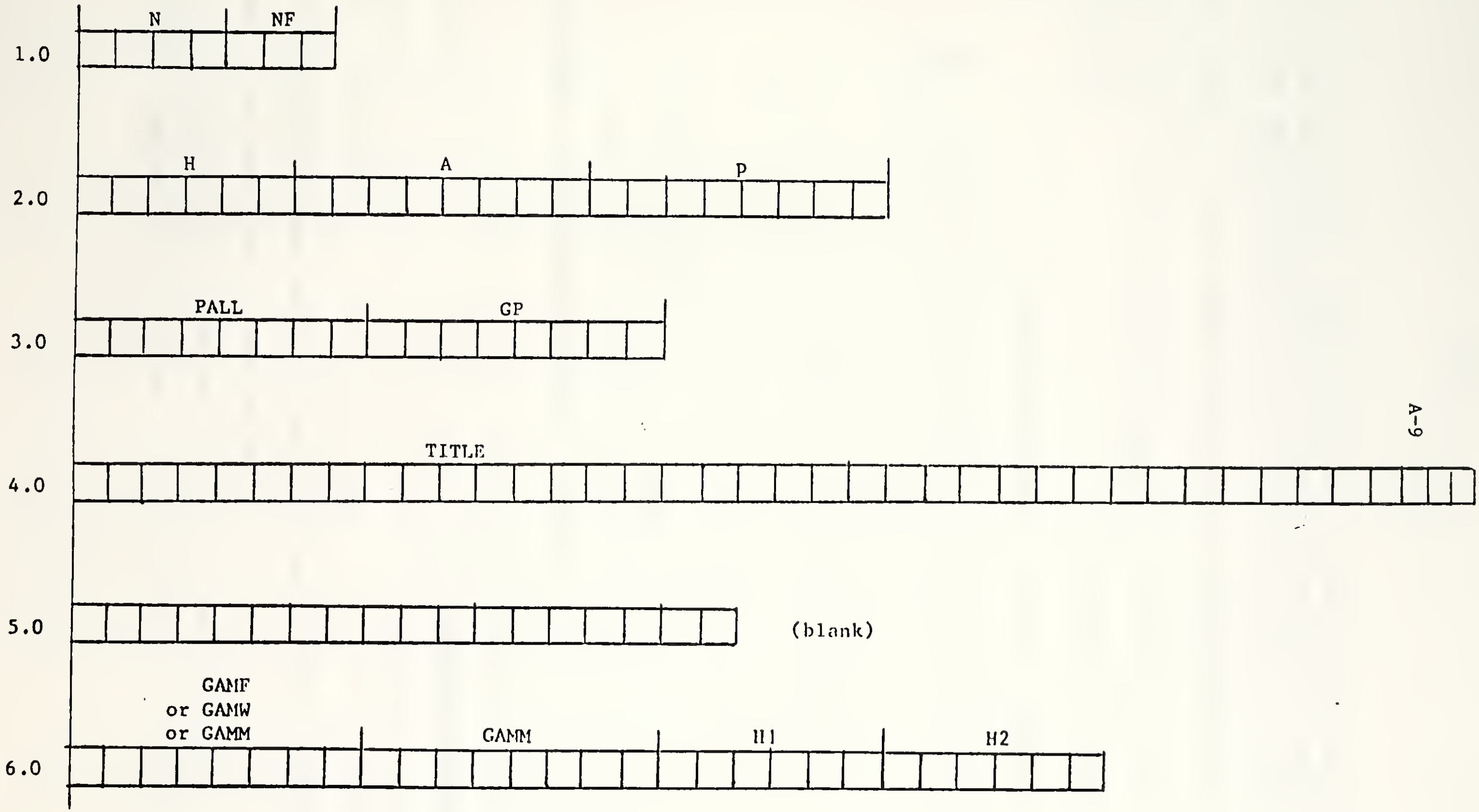
H2 Height of soft layer below fill, to be input for NF=4 only (ft).

The following should be noticed for the input form:

1. N and NF are input on the first card. They should be integers.

INPUT FORM FOR PPILENF

Case:



2. H, A, p are input on the second card. They are real variables. For NF=4, no value of H is provided (blank spaces).

3. PAll, GP are input on the third card.

4. The title (TITLE) is input on the fourth card. It must not exceed 40 characters.

5. The fifth card should be left blank.

6. The input of the sixth card depends on the value of NF:

   NF=1: GAMF is the only input

   NF=2: GAMW is the only input

   NF=3: GAMM is the only input

   NF=4: GAMFF, GAMM, H1, H2 should be input

The input form is given in the following pages, followed by four illustrative examples.

Illustrative Examples:

Example (1):

A pile group is required to carry 290.0 tons. The 16 in. piles used can transfer an allowable load of 60.0 tons each. The piles are to penetrate an 8.0 ft layer of dry recent fill with unit weight 90.0 pcf (Fig. A.1.1). This layer is expected to create negative skin friction on the pile group. It is required to design the pile group to satisfy the negative friction requirements (assuming that all other group requirements are satisfied).

<u>Solution</u>:

No. of piles, assuming no negative friction $= \frac{290.0}{60.0} = 4.83$.

As a first trial, use a group of six piles, in two rows.

Spacing C.L. to C.L between piles = 4.0 ft.

The external group dimensions are 8.33 ft x 5.33 ft.

Area of group = 44.4 sq. ft.

Perimeter of group = 27.3 ft.

The input form (1-1) given is prepared, and a PPILENF run is performed. The output of the run is given.

It can be seen that each pile will be carrying a load of 65.229 tons if the negative friction takes effect. Since this value is an upper bound, the choice can be considered satisfactory. The designer, however, may change the number of piles in this case if he does not allow the pile to carry more than 60.0 tons.

<u>Example</u> (<u>2</u>):

A column load of 275.00 tons is to be carried by a pile group of the same type used in example (1). A maximum groundwater lowering of 6.0 ft is expected. As a result the effective weight of the soil would increase, causing negative friction to be transferred to the pile group.

Solution:

$$\text{No. of piles} = \frac{275}{60} = 4.6 \text{ piles}$$

Assume 6 piles arranged the way described in example (1). The input form (2-1) is shown. PPILENF analysis shows that each pile would be carrying only 47.2 tons, including the anticipated additional load due to negative skin friction. This is not economical since the allowable pile load is 60.0 tons.

Another trial was undertaken with a group of five piles, four in the corners of a square and one in the center (Fig. A.1.2). The external dimensions of the group is 6.33 ft x 6.33 ft. Hence:

Area of pile group = 40.0 sq. ft.

Perimeter of the group = 25.32 ft.

The input form for this trial (2-2) is shown.

The analysis predicted that the load carried by each pile including negative friction would be 56.50 tons, which is close to the allowable load of 60.0 tons. Therefore, this assumption

INPUT FORM FOR PPILENF

Case: *Example (1-1)*

1.0

N	NF
6	1

2.0

H	A	P
8.0	44.40	27.30

3.0

PALL	GP
60.0	290.0

4.0

TITLE
EXAMPLE 1 : CASE OF DRY FILL

5.0 (blank)

6.0

GAMF or GAMW or GAMM	GAMM	H1	H2
90.0			

PILE LOADS DUE TO NEGATIVE SKIN FRICTION

EXAMPLE 1: CASE OF DRY FILL

NUMBER OF PILES = 6

GROUP PERIMETER = 27,300 ft

ALLOWABLE PILE LOAD = 60,000 ton

GROUP LOAD = 290,000 ton

AREA OF PILE GROUP = 44,400 sq ft

DRY DENSITY OF FILL = 90,000 lb/cf

HEIGHT OF FILL = 8,000 ft

RESULTS:

TOTAL GROUP LOAD INCLUDING NEGATIVE FRICTION = 391,376 ton

LOAD CARRIED BY EACH PILE = 65,229 ton

*The assumed number of piles is OK

INPUT FORM FOR PPILENF

Case: Example (2-1)

Card	Fields
1.0	N: 6; NF: 2
2.0	H: 6.0; A: 44.40; P: 27.30
3.0	PALL: 60.0; GP: 275.0
4.0	TITLE: EXAMPLE 2 8 GROUNDWATER LOWERING
5.0	(blank)
6.0	GAMF or GAMW or GAMM: 62.40; GAMM; H1; H2

PILE LOADS DUE TO NEGATIVE SKIN FRICTION

EXAMPLE 2: GROUNDWATER LOWERING

NUMBER OF PILES = 6

GROUP PERIMETER = 27,300 ft

ALLOWABLE PILE LOAD = 60,000 ton

GROUP LOAD = 275,000 ton

AREA OF PILE GROUP = 44,400 sq ft

UNIT WEIGHT OF WATER = 62,400 lb/cf

HEIGHT OF DEWATERING = 6,000 ft

RESULTS:

TOTAL GROUP LOAD INCLUDING NEGATIVE FRICTION = 283,312 ton

LOAD CARRIED BY EACH PILE = 47,219 ton

*Decrease number of piles and repeat the try

INPUT FORM FOR PPILENF

Case: *Example (2-2)*

Line	Fields	Entries
1.0	N, NF	5, 2
2.0	H, A, P	6.0, 40.0, 25.320
3.0	PALL, GP	60.0,
4.0	TITLE	EXAMPLE 2 8 GROUNDWATER LOWERING
5.0	(blank)	
6.0	GAMF or GAMW ✓ or GAMM, GAMM, H1, H2	62.40, , ,

PILE LOADS DUE TO NEGATIVE SKIN FRICTION

EXAMPLE 2: GROUNDWATER LOWERING

NUMBER OF PILES = 5

GROUP PERIMETER = 25,320 ft

ALLOWABLE PILE LOAD = 60,000 ton

GROUP LOAD = 275,000 ton

AREA OF PILE GROUP = 40,000 sq ft

UNIT WEIGHT OF WATER = 62,400 lb/cf

HEIGHT OF DEWATERING = 6,000 ft

RESULTS:

TOTAL GROUP LOAD INCLUDING NEGATIVE FRICTION = 282,488 ton

LOAD CARRIED BY EACH PILE = 56,498 ton

*The assumed number of piles is OK

is acceptable.

Example (3):

It is required to design a pile group for the same conditions given in example (1), except that a 15.0 ft of a very soft clay layer is penetrated by the pile (instead of the fill layer). The clay layer has a saturated unit weight of 115.2 pcf (effective unit weight is 52.8 pcf since the layer is submerged).

Solution:

The input form (3.1) is prepared, and a PPILENF run is performed. The output shows that the assumed number of piles is satisfactory. However, if the designer can arrange a five-pile group with reasonable pile load (not much higher than 60.0 tons), that arrangement would be better from the economical point of view.

Example (4):

A pile group is required to carry a load of 130.0 tons. The top 12.0 ft is formed of an organic and very soft varved silty clay, with the G.W.T. at the ground surface. The saturated unit weight of this layer is 111.5 pcf (effective unit weight is 49.1 pcf). Since it would be very difficult for the pile contractor to do the job because of the very soft layer, a working platform of fill of 6.0 ft thickness (estimated unit weight of 89.0 pcf) would be needed. Piles of 16.0 in. diameter are to be used.

INPUT FORM FOR PPILENF

Case: Example (3-1)

Card	Field	Value
1.0	N	6
	NF	3
2.0	H	15.0
	A	44.40
	P	27.30
3.0	PALL	60.0
	GP	290.0
4.0	TITLE	EXAMPLE 3 : SOFT CLAY LAYER
5.0		(blank)
6.0	GAMF or GAMW or GAMM ✓	52.80
	GAMM	
	H1	
	H2	

PILE LOADS DUE TO NEGATIVE SKIN FRICTION

EXAMPLE 3: SOFT CLAY LAYER

NUMBER OF PILES = 6

GROUP PERIMETER = 27,300 ft

ALLOWABLE PILE LOAD = 60,000 ton

GROUP LOAD = 290,000 ton

AREA OF PILE GROUP = 44,400 sq ft

EFFECTIVE UNIT WEIGHT OF LAYER = 52,800 pcf

HEIGHT OF SOFT LAYER = 15,000 ft

RESULTS:

TOTAL GROUP LOAD INCLUDING NEGATIVE FRICTION = 307,582 ton

LOAD CARRIED BY EACH PILE = 51,264 ton

*The assumed number of piles is OK

They are to be spaced 4.0 ft C.L. to C.L.

<u>Solution</u>:

$$\text{No. of piles} = \frac{130}{40} = 3.25$$

Try a group of four piles (group area = 28.4 sq ft, group perimeter = 21.32 ft) (Fig. A.1.3). It should be noticed that negative skin friction is anticipated due to both the consolidation of the very soft layer and the settlement of the fill layer due to any surficial loads, or submergence.

The input form (4.1) is prepared and a PPILENF run is performed.

The results showed that the four piles would not support the total load, including the anticipated negative skin friction.

Another trial is performed using a group of five piles arranged in the same manner described in examples (2) (2nd trial). The input form (4.2) is shown. This time the analysis showed that the five-pile group would be sufficient for the purpose.

<u>Example</u> (<u>5</u>):

A 40.0 ft high embankment is constructed over a soft clay layer 20.0 ft thick with saturated unit weight of 123.4 psf. This layer is followed by a thick deposit of very stiff clay. A structure is to be constructed over the embankment, but the loads

Case: *Example (4-1)*

Card	Fields and entries
1.0	N: 4 ; NF: 4
2.0	H: (blank) ; A: 28.40 ; p: 21.320
3.0	PALL: 40.0 ; GP: 130.0
4.0	TITLE: EXAMPLE 4: FILL OVER SOFT LAYER
5.0	(blank)
6.0	GAMF ✓ or GAMW or GAMM: 89.0 ; GAMM: 49.10 ; H1: 6.0 ; H2: 12.0

PILE LOADS DUE TO NEGATIVE SKIN FRICTION

EXAMPLE 4: FILL OVER SOFT LAYER

NUMBER OF PILES = 4

GROUP PERIMETER = 21,320 ft

ALLOWABLE PILE LOAD = 40,000 ton

GROUP LOAD = 130,000 ton

AREA OF PILE GROUP = 28,400 sq ft

EFFECTIVE UNIT WEIGHT OF FILL = 89,000 pcf

HEIGHT OF FILL = 6,000 ft

EFFECTIVE UNIT WEIGHT OF SOFT LAYER = 49,100 pcf

HEIGHT OF SOFT LAYER = 12,000 ft

RESULTS:

TOTAL GROUP LOAD INCLUDING NEGATIVE FRICTION = 182,251 ton

LOAD CARRIED BY EACH PILE = 45,563 ton

*Increase the number of piles and repeat the try

INPUT FORM FOR PPILENF

Case: *Example (4-2)*

Card	Fields
1.0	N: 5; NF: 4
2.0	H: ; A: 40.0; P: 25.320
3.0	PALL: 40.0; GP: 130.0
4.0	TITLE: EXAMPLE 4 8 FILL OVER SOFT LAYER
5.0	(blank)
6.0	GAMF ✓ or GAMW or GAMM: 89.0; GAMM: 49.10; H1: 6.0; H2: 12.0

PILE LOADS DUE TO NEGATIVE SKIN FRICTION

EXAMPLE 4: FILL OVER SOFT LAYER

NUMBER OF PILES = 5

GROUP PERIMETER = 25,320 ft

ALLOWABLE PILE LOAD = 40,000 ton

GROUP LOAD = 130,000 ton

AREA OF PILE GROUP = 40,000 sq ft

EFFECTIVE UNIT WEIGHT OF FILL = 89,000 pcf

HEIGHT OF FILL = 6,000 ft

EFFECTIVE UNIT WEIGHT OF SOFT LAYER = 49,100 pcf

HEIGHT OF SOFT LAYER = 12,000 ft

RESULTS:

TOTAL GROUP LOAD INCLUDING NEGATIVE FRICTION = 191,969 ton

LOAD CARRIED BY EACH PILE = 38,394 ton

*The assumed number of piles is OK

were heavy enough that pile foundations were required to support them. A group of 6 piles, 16 in. diameter (same arrangement given in Fig. A.1.3) with allowable load of 55.0 tons each is to be used such that the piles would rest on the very stiff clay layer (Fig. A.1.4). It is required to check the adequacy of such group if the total load = 300.0 tons. The G.W.T. is assumed to at the top of the soft layer.

Solution:

Using the same arrangement used in Example (1) ($A = 44.40$ ft^2, $p = 27.30$ ft), the additional load due to negative skin friction can be checked. It should be noticed that the embankment fill given in this problem cannot be treated the same way as in Example (4). The 40.0 ft high embankment is likely to be compacted with good control. Hence, the long term settlement that might cause negative skin friction can be minimized. Some measures can be taken to reduce the risk of potential problems as follows:

1. The embankment should be compacted using water contents at the wet side of optimum which results in smaller long term deformations due to saturation, etc., than what would occur if the same density was achieved at the dry side of optimum.

2. Stage construction and/or sand drains can be used to facil-

itate the consolidation process before the construction of the piles and the application of other loads.

Details about these procedures are beyond the scope of this report. However, one can assume that if good procedures were followed for the embankment construction, an upper bound value of the additional load due to negative skin friction could be calculated based on the weight enclosed within the soft layer only.

An input form of the data is shown and the output of the analysis indicates the adequacy of the proposed six-pile group.

INPUT FORM FOR PPILENF

Case: Example (5-1)

Card	Fields
1.0	N: 6 · NF: 3
2.0	H: 20.0 · A: 44.40 · P: 27.30
3.0	PALL: 55.0 · GP: 300.0
4.0	TITLE: EXAMPLE 5 : EMBANKMENT OVER SOFT LAYER
5.0	(blank)
6.0	GAMF or GAMW or GAMM: 61.0 · GAMM: · H1: · H2:

PILE LOADS DUE TO NEGATIVE SKIN FRICTION

EXAMPLE 5: EMBANKMENT OVER SOFT LAYER

NUMBER OF PILES = 6

GROUP PERIMETER = 27,300 ft

ALLOWABLE PILE LOAD = 55,000 ton

GROUP LOAD = 300,000 ton

AREA OF PILE GROUP = 44,400 sq ft

EFFECTIVE UNIT WEIGHT OF LAYER = 61,000 pcf

HEIGHT OF SOFT LAYER = 20,000 ft

RESULTS:

TOTAL GROUP LOAD INCLUDING NEGATIVE FRICTION = 327,084 ton

LOAD CARRIED BY EACH PILE = 54,514 ton

*The assumed number of piles is OK

PPILENF LISTING

Program Characteristics:

Core Used = 046000 B octal words

Time Used = 0.447 CPU seconds

NL = 7300 words

CX = 0.966 sec

```
      program main (tape5,tape6)
c
c
c
c    ************************************************************
c    *                                                          *
c    * this program computes the additional pile loads due to  *
c    * negative skin friction                                   *
c    *                                                          *
c    ************************************************************
c
c
c
      dimension title(20)
c
c
      read (5,1000)n,nf
 1000 format(i4,i3)
      read (5,2000)h,a,p
 2000 format(f6.3,f8.3,f8.3)
      read (5,3000)pall,gp
 3000 format(2f8.3)
c
c
c
      read (5,111)title
  111 format(10a4)
c
c
      write (6,2)
    2 format(///1x,'purdue university',
     &/25x,'pile loads due to negative skin friction')
      write(6,99)title
   99 format(////14x,10a4)
c
c
c
      write (6,88) n,p,pall,gp,a
   88 format(//10x,18hnumber of piles = ,i3,
     &//10x,18hgroup perimeter = ,f8.3,3x,3hft.,
     &//10x,22hallowable pile load = ,f8.3,3x,3hton,
     &//10x,13hgroup load = ,f8.3,3x,3hton,
     &//10x,21harea of pile group = ,f8.3,3x,6hsq.ft.)
c
c
c
      if(nf.eq.1)goto 10
      if(nf.eq.2)goto 20
      if(nf.eq.3)goto 30
      if(nf.eq.4)goto 40
c
c
c     case of dry fill layer subject to future settlement
c
   10 read(5,3)gamf
    3 format (f8.3)
      a1=a+h*p+4.0*h**2
      gpnf=gamf*h*(a+a1)/(2.0*2000.0)
      write (6,77)gamf,h
```

```
 77 format(//10x,22hdry density of fill = ,f8.3,3x,7hlb./cf.,
   &//10x,17hheight of fill = ,f6.3,3x,3hft.)
    goto 100


    case of ground water lowering

 20 read(5,4)gamw
  4 format(f8.3)
    gpnf=gamw*h*a/2000.0
    write (6,66)gamw,h
 66 format(//10x,23hunit weight of water = ,f8.3,3x,7hlb./cf.,
   &//10x,23hheight of dewatering = ,f6.3,3x,3hft.)
    goto 100


    case of consolidation of soft clay or remoulding of sesitive clay

 30 read (5,5)gamm
  5 format(f8.3)
    gpnf=gamm*h*a/2000.0
    write(6,55)gamm
 55 format(//10x,'effective unit weight of layer = ',f8.3,3x,'pcf.')
    write(6,551)h
551 format(//10x,'height of soft layer = ',f6.3,3x,'ft.')
    goto 100


    case of fill overlying consolidating soft clay

 40 read(5,6)gamf,gamm,h1,h2
  6 format(2f8.3,2f6.3)
    a1=a+h1*p+4.00*h1**2
    gpnf1=gamf*h1*(a+a1)/(2.0*2000.0)
    gpnf2=gamm*h2*a/2000.0
    gpnf=gpnf1+gpnf2
    write(6,44)gamf,h1,gamm,h2
 44 format(//10x,'effective unit weight of fill = ',f8.3,3x,'pcf',
   &//10x,'height of fill = ',f6.3,3x,'ft.',
   &//10x,'effective unit weight of soft layer = ',f8.3,3x,'pcf'
   &//10x,23hheight of soft layer = ,f6.3,3x,3hft.)



    check of pile loads

100 gpt=gp+gpnf

    write (6,999)
999 format(////2x,'results:')

    write(6,33)gpt
 33 format(///10x,'total group load including negative friction
   & = ',f8.3,3x,'ton')

    pt=gpt/n
    write(6,7)pt
  7 format(///10x,27hload carried by each pile = ,f8.3,3x,3hton)

    pt1=1.10*pall
```

COVER DESIGN BY ALDO GIORGINI